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	Engineering and Design STRUCTURAL DESIGN OF CLOSURE STRUCTURES FOR LOCAL FLOOD PROTECTION PROJECTS	
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Engineering and Design
STRUCTURAL DESIGN OF CLOSURE STRUCTURES
FOR LOCAL FLOOD PROTECTION PROJECTS

1. Purpose. This engineer manual (EM) provides guidance for the structural design of closure structures for openings in levees and floodwalls of inland local flood protection projects (LFPPs). Design requirements for aluminum and steel stoplog closure structures and steel swing, miter, trolley, and rolling gate closure structures for openings in levees and floodwalls of LFPPs are addressed. This guidance provides a framework for incorporating the required elements of design execution into the design process for closure structures.

2. Applicability. This manual is applicable to all HQUSACE elements, major subordinate commands, districts, laboratories and field operating activities having civil works design and construction responsibilities.

FOR THE COMMANDER:

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Engineering and Design
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FOR LOCAL FLOOD PROTECTION PROJECTS**

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Chapter 1
Introduction

1-1. Purpose and Scope

a. Purpose. This engineer manual (EM) provides guidance for the structural design of closure structures for openings in levees and floodwalls of inland local flood protection projects (LFPPs).

b. Scope. The scope of this guidance includes the design requirements for aluminum and steel stoplog closure structures and steel swing, miter, trolley, and rolling gate closure structures for openings in levees and floodwalls of LFPPs. This guidance provides a framework for incorporating the required elements of design execution into the design process for closure structures.

1-2. Applicability

This manual is applicable to all HQUSACE elements, major subordinate commands, districts, laboratories, and field operating activities having civil works design and construction responsibilities.

1-3. References

a. EM 1110-2-38, Environmental Quality in Design of Civil Works Projects.

b. EM 1110-2-301, Guidelines for Landscape Planting at Flood Walls, Levees, and Embankment Dams.

c. EM 1110-2-2104, Strength Design for Reinforced-Concrete Hydraulic Structures.

d. EM 1110-2-2105, Design of Hydraulic Steel Structures.

e. EM 1110-2-2502, Retaining and Flood Walls.

f. EM 1110-2-2703, Lock Gates and Operating Equipment.

g. EM 1110-2-2906, Design of Pile Foundations.

h. EM 1110-2-3104, Structural and Architectural Design of Pumping Stations.

i. EM 1110-2-3400, Painting: New Construction and Maintenance.

j. Aluminum Association, Inc. 1986. "Specifications for Aluminum Structures," 900 19th Street, NW, Suite 300, Washington, DC 20006.

k. American Association of State Highway and Transportation Officials (AASHTO). 1989. "Standard Specifications for Highway Bridges," 14th ed., 444 North Capitol Street, NW, Suite 225, Washington, DC 20001.

l. American Institute of Steel Construction (AISC). 1986. "Manual of Steel Construction, Load and Resistance Factor Design (LRFD)," 1st ed., 400 North Michigan Avenue, Chicago, IL 60611.

m. American Institute of Steel Construction (AISC). 1989. "Manual of Steel Construction, Allowable Stress Design (ASD)," 9th ed., 400 North Michigan Avenue, Chicago, IL 60611.

n. American Railway Engineering Association. 1991. "1991 Manual for Railway Engineering," Volume I, Chapters 1 through 8, and Volume II, Chapters 9 through 33, 50 F Street, NW, Suite 7702, Washington, DC 20001.

1-4. Background

This manual was developed to provide uniform criteria for the design of closure structures for openings in levees and floodwalls of LFPPs. The development process involved reviewing and evaluating the design and performance of existing closure structures constructed throughout the Corps to identify the structures which are cost-effective and efficient in operation, and incorporating the design features of these projects into the criteria provided herein. The resulting guidance contained in this manual provides a series of design requirements and provisions that should be applied to the design of closure structures.

Chapter 2
Design

2-1. Design Process

The structural design of closure structures for openings in levees and floodwalls should be planned in detail to ensure that the elements of the structural design responsibility are properly executed during the reconnaissance phase through the operations phase of projects. These elements include: coordinating the required input from the project manager, project engineer, and other engineering disciplines and functional members of the design team; incorporating general design provisions into the design process and resulting solutions which pertain to operational, functional, maintenance, aesthetics, safety, security, construction, and economical considerations; examining and comparing pertinent features of closure types, selecting the appropriate closures and suitable alternatives for particular applications, and defining the preferred general design concept which provides a basis for estimating the construction cost; and performing the required structural analyses and design of structural features and ensuring design quality through design review and other methods of verification. Appendix A provides a checklist of design functions required during each phase of the design process.

2-2. Design Coordination

Full coordination of the total design process should be considered as the most essential element for ensuring design quality. The structural engineer must operate within the prescribed role and functional relationships established between the structural discipline and the project manager, project engineer, engineering disciplines, and other functional elements engaged in the design process to effectuate the coordination required for the design of closures.

a. Project manager. Throughout the design process, the project manager is responsible for coordinating with the local sponsor, communicating input from the local interests to the project engineer and design engineers, and ensuring that the major design features of closure structures are acceptable to the local interests. The project manager must also obtain agreement with agencies or entities which own and maintain utilities and facilities which would be impacted by the construction of the closure structures. Such facilities include pipe lines, transmission lines, streets, highways, bridges, and railroads.

b. Project engineer. The project engineer, in managing the design of closure structures, is responsible for interacting with the project manager to obtain the agreed local sponsor requirements and acceptance of proposed design solutions for closure structures and communicating these to the functional elements engaged in the project including real estate, planning, engineering, construction, and operations.

c. Structural engineer. The structural engineer is responsible for coordinating with the project engineer and other engineering disciplines engaged in the design of closure structures, including civil, geotechnical, hydraulic, mechanical, coastal, environmental, construction, and operations engineers, to ensure all pertinent engineering considerations are properly integrated into the structural design.

2-3. General Design Provisions

General design provisions which must be integrated into the design process for the design and construction of closure structures for LFPP include operational, functional, maintenance, aesthetics, safety, security, construction, and economical considerations. These provisions must be fully and appropriately addressed in the design process and the resulting project documents.

a. Operational. The required lead time for closing the structures in the event of flooding is controlled by the rate of rise of the flood waters. The accuracy of advanced forecasting of flooding determines the length of time which should be scheduled for the mobilization of operations personnel and equipment required to close the structures. Also, the accuracy of forecasting determines the number of times traffic is inconvenienced by a closed structure when flooding does not occur. The scheduled time for closure of structures should include a cushion to ensure that closure is completed well before the arrival of flood waters. The required lead time and the types of operating equipment and operations personnel available to the local sponsor are the primary factors in determining the type of closure structures suitable for particular applications.

b. Functional. Functional requirements for facilities, such as roadways and railroads, affect the design of closure structures. Existing site topography and clearance requirements for roadways (highways and streets) and railroads are primary functional considerations which must be incorporated into the design of closure structures.

(1) Site topography. Existing grades parallel to roadways and railroads affect the required site distances along these facilities. Variation of grades along roadways and railroads affects the selection of the type of closure structures and the design of sills and supports for these facilities. For example, multiple sets of railroad tracks occurring at different grade elevations require a hinged bottom seal arrangement to provide an effective seal.

(2) Clearances.

(a) Roadways. The opening widths provided for roadways shall comply with the requirements of the American Association of State Highway and Transportation Officials (AASHTO) (1989). The width of closure openings should not be less than 30 ft for roadways with two lanes of traffic. The minimum vertical clearance between the crown of roadways and fixed overhead components of closures should not be less than 14 ft. Clearances should be coordinated and approved by the facility owner.

(b) Railroads. Minimum horizontal and vertical clearances shall not be less than that required by the AASHTO (1989). The normal minimum width of opening provided for railroads is approximately 20 ft for each set of tracks involved in the closure. Clearances should be coordinated with and approved by the facility owner.

c. Maintenance. Proper maintenance of closure structures is essential to the continuous satisfactory performance of the structures. The required maintenance provisions must be included in the agreement with the local sponsor. Current agreements with local sponsors require annual periodic inspections of the closure structures and the adjoining levee or floodwall. Inspections must be thorough so that any deficiencies that are critical to the function of the project are detected and promptly corrected. Designs should incorporate materials, systems, and features which are economically feasible and require minimal maintenance.

d. Aesthetics. The requirements for incorporating aesthetic quality into the design of LFPP have been established. EM 1110-2-38 provides guidance for aligning flood control channels, landscaping along channels, and the aesthetic treatment of channel linings. EM 1110-2-301 also provides guidance for landscaping. In some LFPP floodwalls, an open view of the waterway was made an aesthetic requirement. These projects incorporate closures with bottom hinged walls which can be stored in the lowered position and raised for protection during flood periods.

e. Safety. The design of closures must include safety provisions for the public and the operations personnel. Local sponsors are responsible for the safe operation of closure structures; therefore, designers must coordinate with sponsors so that the appropriate design provisions are incorporated to ensure safe operation. General safety provisions include providing railings on the top of gates and adjacent walls for public protection and providing ladders for access by operations personnel. Additional safety features could include warning signs and barriers which prevent access by unauthorized persons.

f. Security. The design of closure structures must include security provisions which prevent vandalism and the impairment of operating capability. Locked storage facilities which are inaccessible to the public should be provided for the storage of stoplogs, removable posts, and other unsecured parts of closure structures. In areas subject to vandalism, masonry buildings should be used. Latching devices which hold gates in the stored position should be provided with adequate locks.

g. Construction.

(1) Procedures and methods. Construction procedures and methods should be considered during design to facilitate the general constructibility of closure structures. All phases of construction and erection procedures, particularly for gated closure structures, should be considered and design details developed which minimize complexities.

(2) Assurance of design integrity. The structural designer must identify, in the project documents, the design assumptions, details, and specifications essential to design integrity. This is necessary to make certain that the closure features receive assurance inspection during construction to verify that actual construction methods are in compliance with the design assumptions, details, and specifications.

h. Cost comparisons. The costs of previously constructed closure structures vary according to the closure type and opening size. These variations should be considered in making cost-effective decisions in the selection of the closure type and the design of closure structures for openings in levees and floodwalls of LFPPs.

Chapter 3
Selection of Closure Types

3-1. Facilities Requiring Closures

Closure structures are required at openings in levee and floodwall systems when facilities such as railroads, roadways, and pedestrian walkways pass through levee and floodwall systems at elevations below the level of protection provided by the project. The widths of openings for these facilities vary depending upon their functional purpose. Openings for railroads and roadways vary from moderate widths (20 to 40 ft) to widths of 100 ft or more. Openings for pedestrian walkways are usually 20 ft or less. The need for closures should be avoided when possible by using ramps or pedestrian overpasses.

3-2. Closure Structure Types

Closure structures for openings in levee and floodwall systems of LFPPs are usually either stoplog or gate type closures. Stoplog closure structures are usually the least expensive. Gate closure structures are more functional because they can usually be closed and opened quickly and with less effort.

a. Stoplog closure structures. Stoplog closure structures usually consist of one or more sets of horizontal aluminum or steel beams, stacked vertically in the closed position. Aluminum stoplogs weigh less than steel stoplogs of the same size but do not have the same strength. For narrow openings, one set of beams or logs may span between support slots constructed at the edge of openings. For wider openings, intermediate, removable support posts are required as shown in Figure 3-1. Seals are not normally attached to the stoplogs; however, plastic sheeting, sandbags, or other available means should be used to reduce leakage through the stoplog closure structure. Storage facilities must be provided for the stoplogs, removable posts, and accessories. When secured areas are available, closure items may be stored on uncovered storage concrete pedestals or slabs; otherwise, a storage building must be provided. Typical details of a stoplog closure structure are shown in Plates 1-2. Advantages and disadvantages of stoplog closure structures are given below.
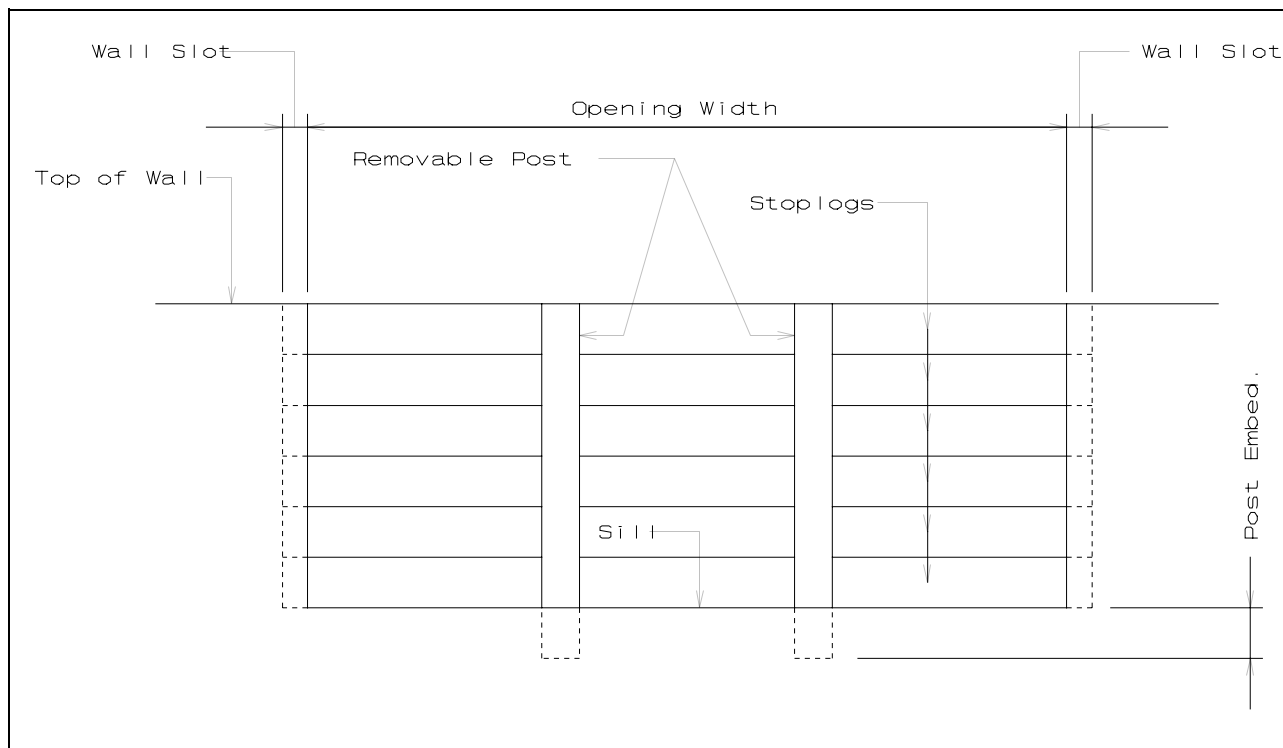

Figure 3-1. Stoplog closure structure

Advantages:

- Fabrication methods are simple and economical.

- Initial cost is usually less than for gate closures.

- Is easily operated for narrow and low openings.

Disadvantages:

- Intermediate support posts or wide stoplogs are required for wide openings which usually require special lifting equipment for installation.

- Accurate long-range weather forecasting is needed since a relatively long lead time is required to mobilize personnel and equipment for installation.

- Installation time is usually increased to allow cleaning of the post sockets during installation.

- Installation time is longer than required to close gated closures.

- A storage building is required to prevent damage by vandalism or loss by theft.

b. Gate closure structures. The most common type of gates used for gate closure structures are swing, miter, rolling, and trolley gates. Figure 3-2 shows outside boundary envelopes for a limited number of gate closure structures from past Corps projects. Gate types are plotted relative to the size of the closure opening. The gate sizes enclosed by the envelopes lie to the left and below the applicable curve or line. Figure 3-2 shows the type of
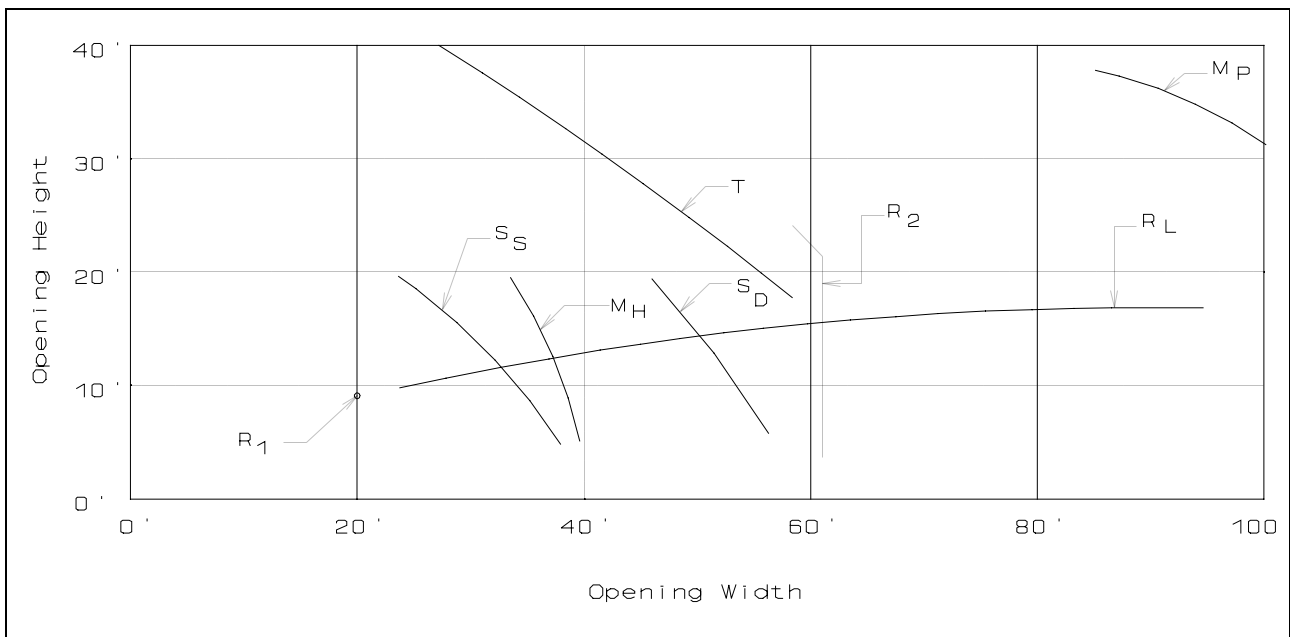
LEGEND

S_S = Swing gate, single leaf.
S_D = Swing gate, double leaf.
M_H = Miter gate with hinges.
M_P = Miter gate with pintle, continuous quoin, and miter posts.
R_2 = Rolling gate stabilized by a double line of wheels.
R_1 = Rolling gate with a single line of wheels, trolley stabilized.
R_L = Rolling gate - L-frame, latch stabilized.
T = Trolley gate.

Figure 3-2. Gate type versus opening dimensions

gate and height-to-width relationships used in past designs and is not intended to establish the maximum opening sizes for any particular gate type used in future designs.

(1) Swing gates. Swing gates are composed of two or more horizontal girders, vertical intercostals, vertical end diaphragms, a skin plate, and diagonal braces. Swing gates are supported on one side by top and bottom hinges attached to a support structure as shown in Figure 3-3. In most cases, swing gate closures consist of a single swing gate leaf. However, double leaf gates are used for wide openings. Double leaf gates must be stabilized by a removable center post or diagonal tie-back linkages as shown in Figure 3-4. One end of the diagonal linkage rods shall be permanently attached to the free ends of each gate leaf. The other end of the each linkage rod is attached to the support structure when the gates are closed. A support jack is provided beneath the gate to withstand the vertical component of load from the linkage rods. Rubber J-seals are attached to gates to form a continuous water-tight seal between the gates and supporting walls and sill of the opening. Closure provisions should include the use of winches or motor vehicles to accomplish closure during strong winds. Typical details of a swing gate closure structure are shown in Plate 3. Typical details of hinges, seals and latches are shown in Plates 4-12. Advantages and disadvantages of swing gate closure structures are given below.

Advantages:

- Single leaf swing gates are more practical for opening widths up to 40 ft.

- Skilled personnel or equipment are not required for operation except when removable intermediate support posts are used with double leaf gates.

- A short lead time is required for making closure except when removable intermediate support posts are used with double leaf gates.

Disadvantages:

- Requires right-of-way area for operating.

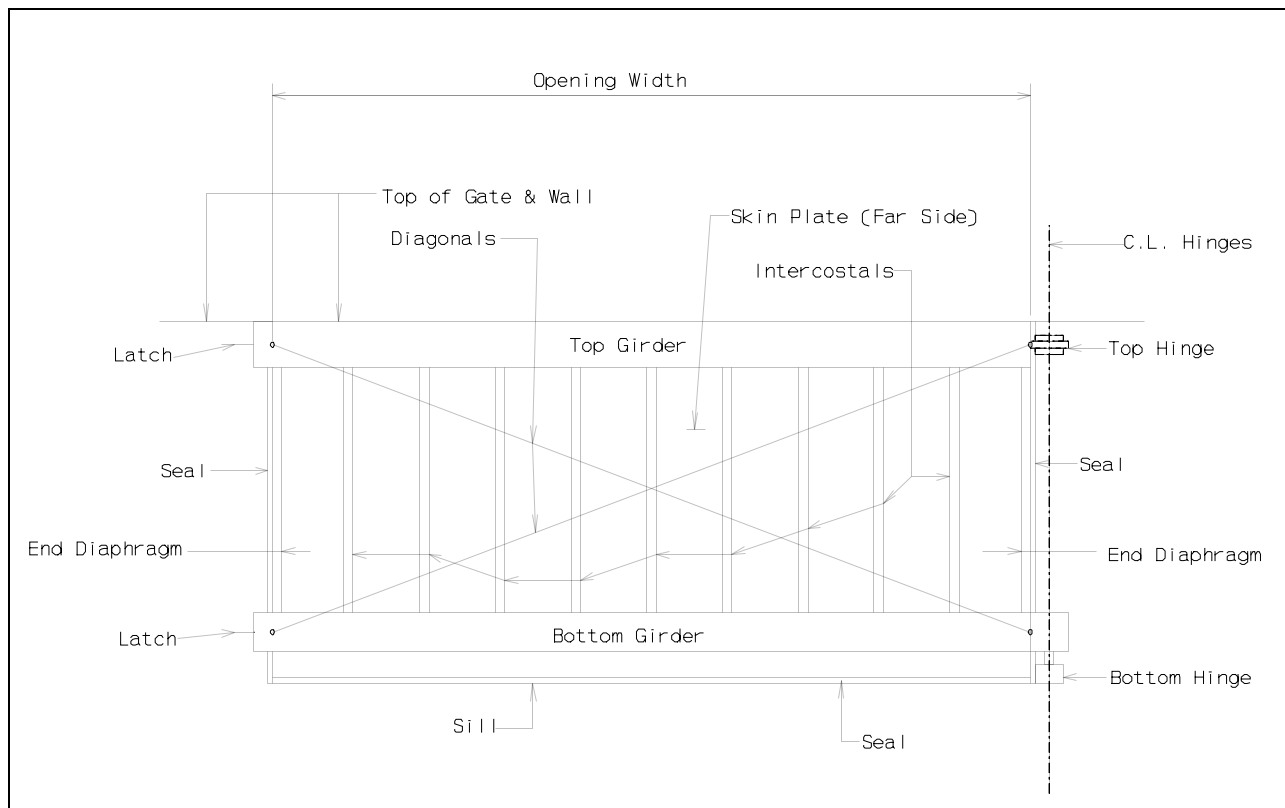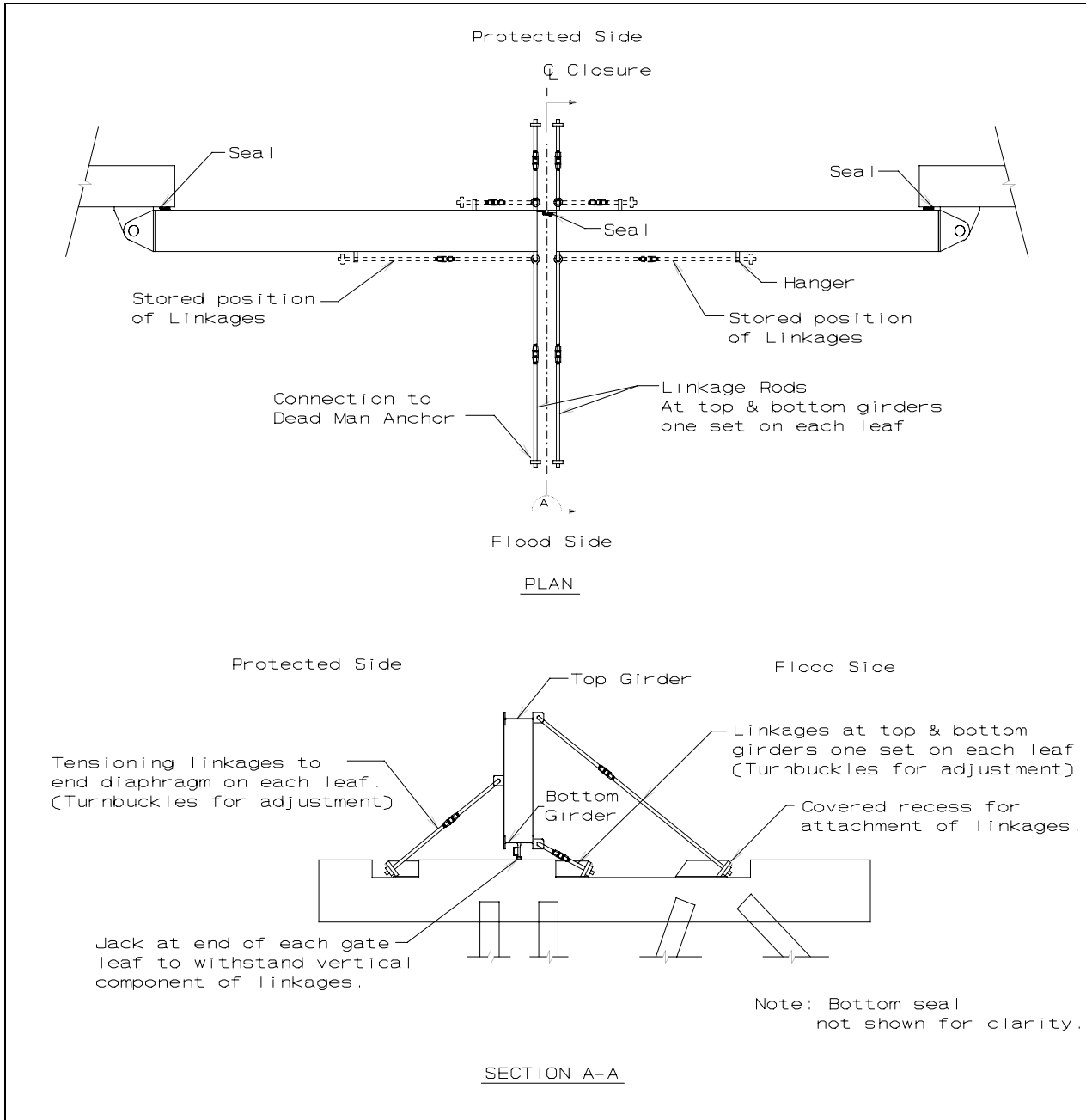- Requires complex shop fabrication with machine work.

Figure 3-3. Swing gate closure structure

```
                              Protected Side

                              C Closure

        Seal                                              Seal
                                                                    Seal

        Stored position                                   Hanger
        of Linkages                             Stored position
                                                of Linkages

                                              Linkage Rods
                                              At top & bottom girders
        Connection to                         one set on each leaf
        Dead Man Anchor

                                   A

                              Flood Side

                              PLAN


        Protected Side                         Flood Side

                              Top Girder

                                              Linkages at top & bottom
                                              girders one set on each leaf
  Tensioning linkages to                      (Turnbuckles for adjustment)
  end diaphragm on each leaf.
  (Turnbuckles for adjustment)    Bottom
                                  Girder       Covered recess for
                                              attachment of linkages.

  Jack at end of each gate
  leaf to withstand vertical
  component of linkages.
                                   Note: Bottom seal
                                         not shown for clarity.

                              SECTION A-A
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Figure 3-4. Tie-back linkage for double leaf swing gate

- A storage facility is required when removable intermediate support posts are used with double leaf gates.

- Requires a retractable bottom sill to accommodate nonlevel sill surfaces.

- Is difficult to operate during high winds.

(2) Miter gates. Miter gates consist of two leaves that form a three-hinged arch when the gates are in the closed position. Each gate leaf is composed of: horizontal girders, vertical intercostals, vertical end diaphragms, a skin plate, and adjustable diagonal tension rods. The gate leaves are attached to support piers by top and bottom hinges as shown in Figure 3-5. The diagonal tensioning
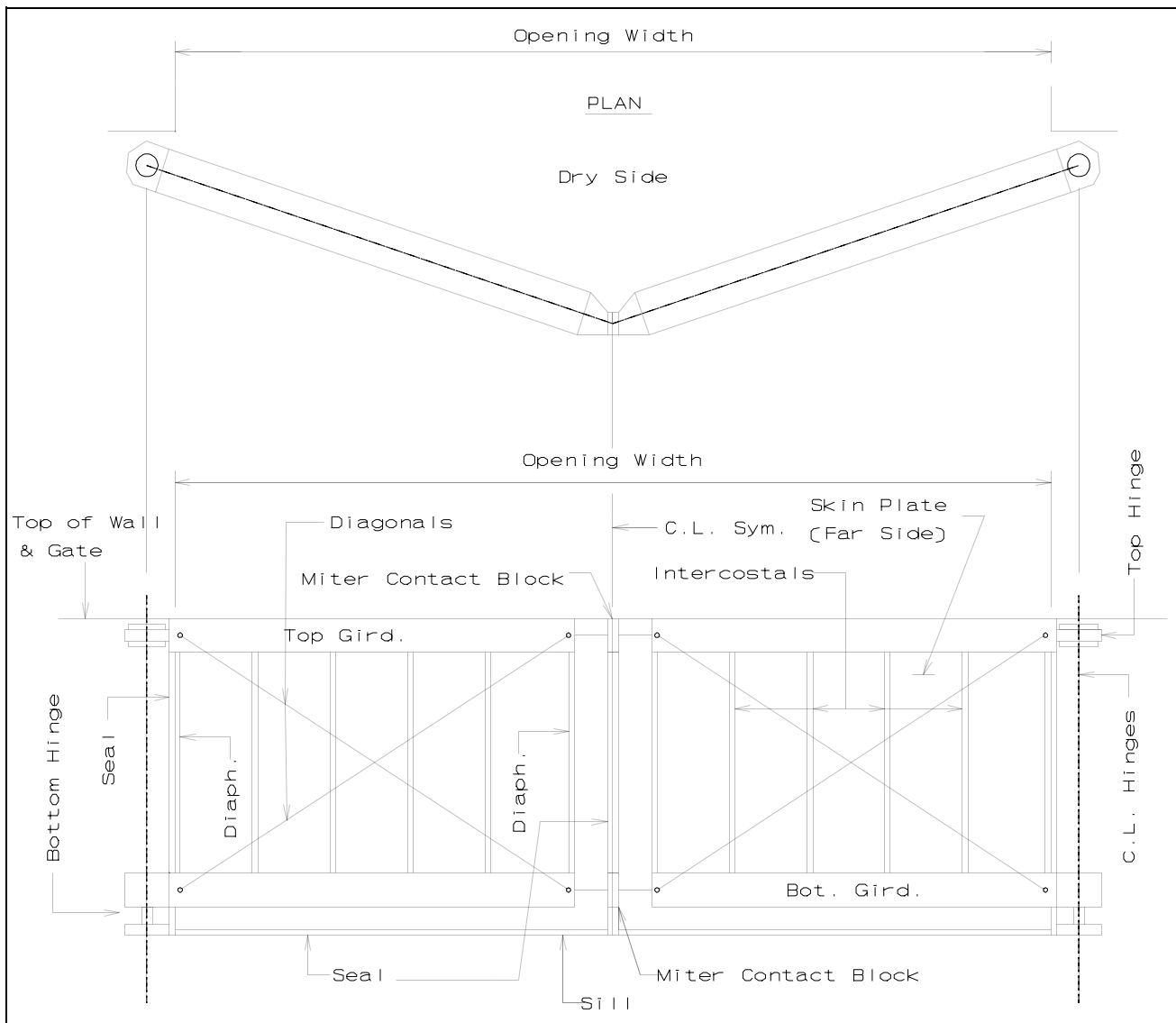
Figure 3-5. Miter gate closure structure

rods are required to prevent twisting of the gate leaves due to their dead load and must be properly tensioned after the gates are installed so that the gates hang plumb and miter properly. Typical diagonal prestressing notes are given in Plate 13. For miter gates with two horizontal girders, the three-hinged arch reactions are resisted by the top and bottom hinges at the supports and spot bearing blocks at the miter ends of the horizontal girders. The magnitude of loading on large miter gates requires the use of three or more horizontal girders, quoin posts with bearings attached to the support piers, and continuous miter posts at the miter ends of the gates to accommodate the forces. Also, hemispherical pintles and top linkages, similar to navigation lock gates, may be required instead

of hinges. Provisions for the design of hemispherical pintles and top linkages are given in EM 1110-2-2703. Hinges and miter blocks or bearing posts must be adjustable to accommodate construction tolerances and allow the gates to miter properly. Support structures for miter gates are usually more difficult to design and cost more than support structures for other types of gates. The supporting structures and their foundations must be designed to minimize the deflections at the gate hinges or quoin posts so that the gates will function as designed. J-seal assemblies are provided for water tightness. Latches are provided to secure the gates in the stored and closed position. Seal, hinge, and latch details for miter gates are similar to those used for swing gates. Closure

provisions should include the use of winches or motor vehicles to accomplish closure during strong winds. Typical details of a miter gate closure structure are shown in Plates 14-16. Advantages and disadvantages of miter gate closure structures are given below.

Advantages:

- Is suitable for large openings.

- Closure can be made quickly without the use of skilled personnel.

- A storage building is not required.

- Weighs less than other types of gates designed for large openings.

- A center support is not required.

Disadvantages:

- Requires complex shop fabrication with machine work.

- Requires right-of-way area for operating.

- Support structure is more complex to design and more expensive than for other gate types.

- Requires a retractable bottom seal to accommodate nonlevel sill surfaces.

- Is difficult to operate during high winds.

(3) Rolling gates. Rolling gates are composed of a structural steel frame covered with a water barrier skin plate. The gates are supported by wheels that roll on tracks embedded in the sill across the closure opening and the storage area. J-seals are attached to the ends and bottoms of the gates to form a water-tight seal between the gates and the plates embedded in the end supports and the bottom sill. The gates are sometimes operated by a cable attached to a truck motorized winch; however, the cable could also be connected directly to a truck which pulls the gate open or closed. Alternately, the design may consist of a winch mounted at the site for gate operations. Gates along fast rising streams may be designed to be opened or closed from the protected side of the floodwall as shown in Plates 17-20. Latches should be provided to secure the gates in the stored and closed positions.

(4) Rolling gate - stabilized with two lines of wheels. Rolling gates stabilized with two lines of wheels are composed of: horizontal girders, vertical intercostals, vertical end and intermediate plate diaphragms, a skin plate, and two lines of support wheels as shown in Figure 3-6. The wheels support and stabilize the gate against overturning. The wheels are usually V-grooved castings and roll on tracks that are usually inverted angles with embedded anchorages. The depth of the bottom girder is usually governed by the required transverse spacing between the supporting wheels rather than the hydrostatic load. A girder depth of 30 to 36 in. is normally required to accommodate the spacing between the two lines of wheels to provide stability of the gate during opening and closing operations. Typical details of a rolling gate stabilized with two lines of wheels are shown in Plate 21. Advantages and disadvantages of rolling gates stabilized with two lines of wheels are given below.

Advantages:

- Is adaptable to wide openings.

- Closure can be made quickly without the use of skilled personnel.

- A storage building is not required.

- Requires small storage space.

Disadvantages:

- Requires a retractable bottom seal to accommodate nonlevel sill surfaces.

- Unless wheel assemblies are designed to accommodate the lateral bottom girder deflection, jacks must be provided to lift the wheel assemblies from the tracks when the gate is in the closed position.

- Requires level storage area immediately adjacent to the closure opening.

(5) Rolling gate - with single line of wheels and stabilizing trolleys. These gates are usually composed of a trussed steel frame covered with skin plate or bridge planks. The gates are supported at the bottom by a single line of wheels and are stabilized laterally by an extended top girder supported by trolleys attached to the top of the floodwall as shown in Figure 3-7. Girder depths are usually governed by the hydrostatic loading on the gate.
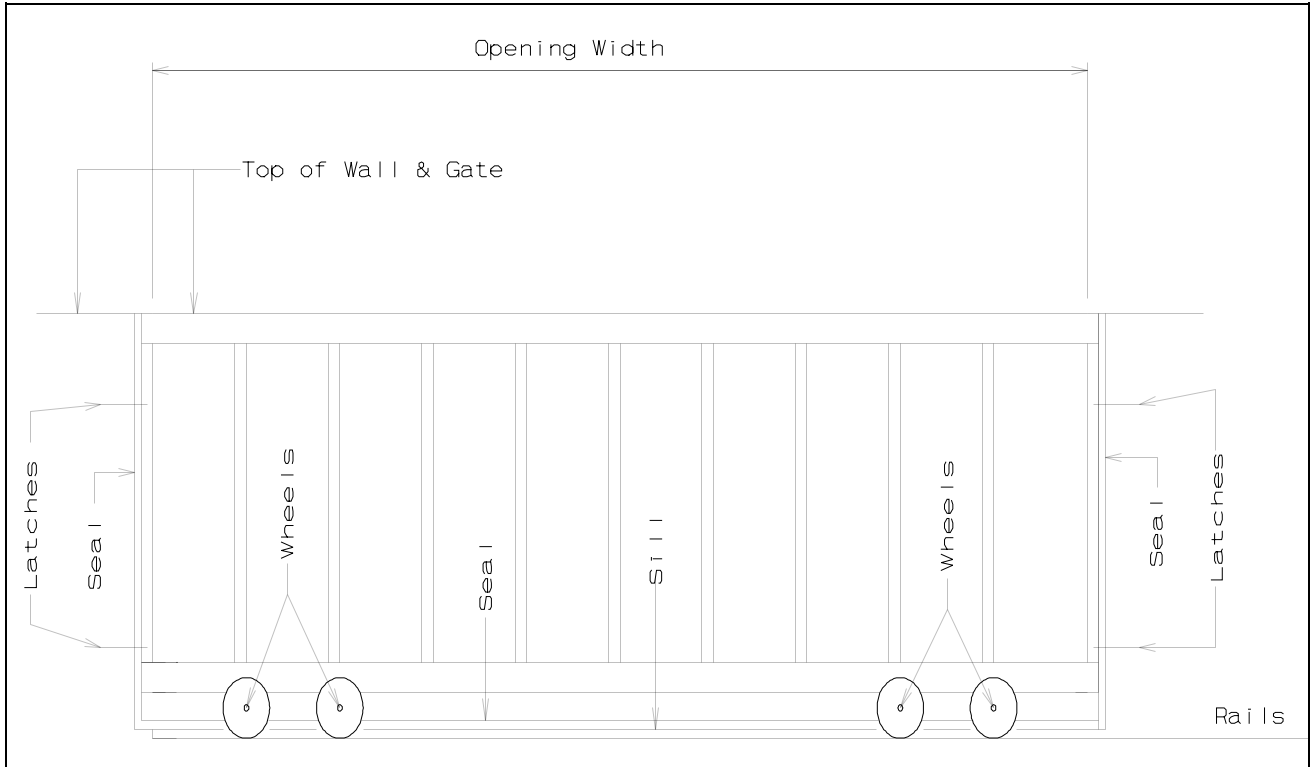
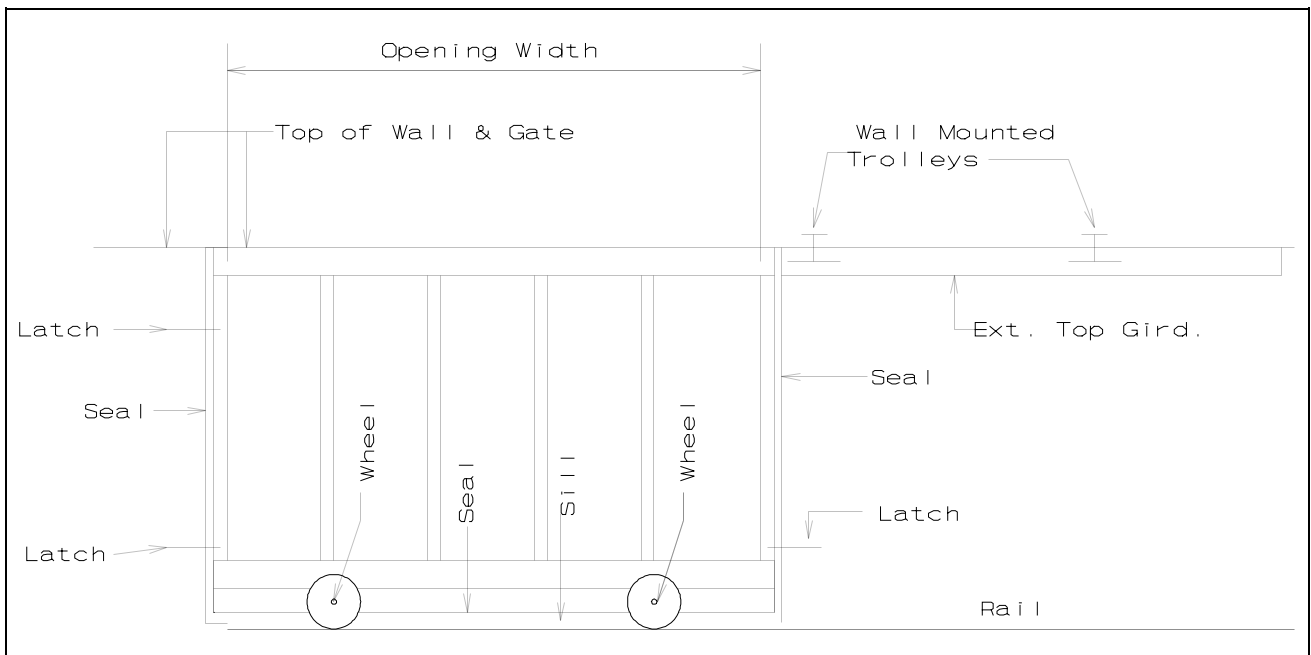**Figure 3-6. Rolling gate - stabilized with two lines of wheels**

Figure 3-7. Rolling gate - single line of wheels and stabilizing trolleys

Typical details of a rolling gate with a single line of wheels and stabilizing trolleys are shown in Plate 22. Advantages and disadvantages of rolling gates with a single line of wheels and stabilizing trolleys are given below.

Advantages:

- Is practical for closure widths up to 30 ft.

- Closure can be made quickly without the use of skilled personnel.

- A storage building is not required.

- Requires small storage space.

Disadvantages:

- Requires a retractable bottom seal to accommodate nonlevel sill surfaces.

- Requires level storage area immediately adjacent to the closure opening.

(6) Rolling gate - L-frame. These gates are usually composed of a series of L-shaped structural steel frames interconnected by horizontal and diagonal members. The gates are supported at the bottom by two lines of wheels as shown in Figure 3-8. Hooks attached to the heel of each of the L-frames engage anchorages embedded in the concrete sill structure to stabilize the gate against hydro-static loadings. Typical details of an L-frame rolling gate are shown in Plate 23. Advantages and disadvantages of L-frame rolling gates are given below.

Advantages:

- Can be designed for any opening width.

- Can be shop-fabricated in sections to simplify handling and storage.

- Closure can be made quickly without the use of skilled personnel.

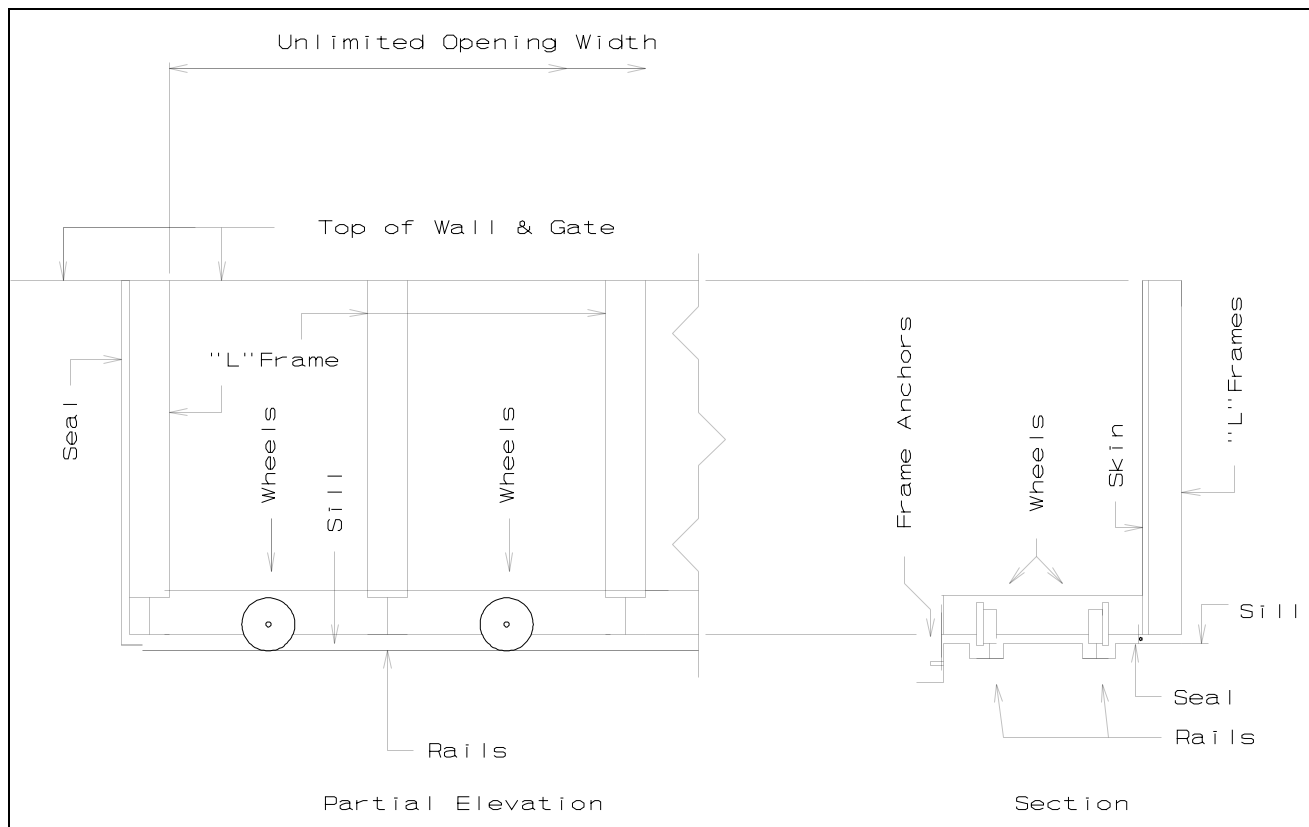- A storage building is not required.

Figure 3-8. Rolling gate - L-frame stabilized by hooks

- Requires small storage space.

Disadvantages:

- Requires a retractable bottom seal to accommodate nonlevel sill surfaces.

- Requires level track surface.

- Requires level storage area immediately adjacent to the closure opening.

- Requires wide sill to accommodate the installation of tracks and hook anchorages.

(7) Trolley Gates. Trolley gates are usually composed of top and bottom horizontal girders, other secondary framing members, and a skin plate. Trolley gates are suspended from trolleys running on an overhead rail and beam supported by the floodwall as shown in Figure 3-9. The gates are opened and closed by a winch arrangement

similar to that used for rolling gates. Typical details of a trolley gate closure structure are shown in Plate 24. Advantages and disadvantages of a trolley gate closure structure are given below.

Advantages:

- Is practical for closure widths up to 60 ft.

- Closure can be made quickly without the use of skilled personnel.

- Can obtain a good seal against irregular sill surfaces.

- A storage building is not required.

- Requires small storage space.

- Is suitable for railroad closures because required vertical clearances for railroads are fixed.
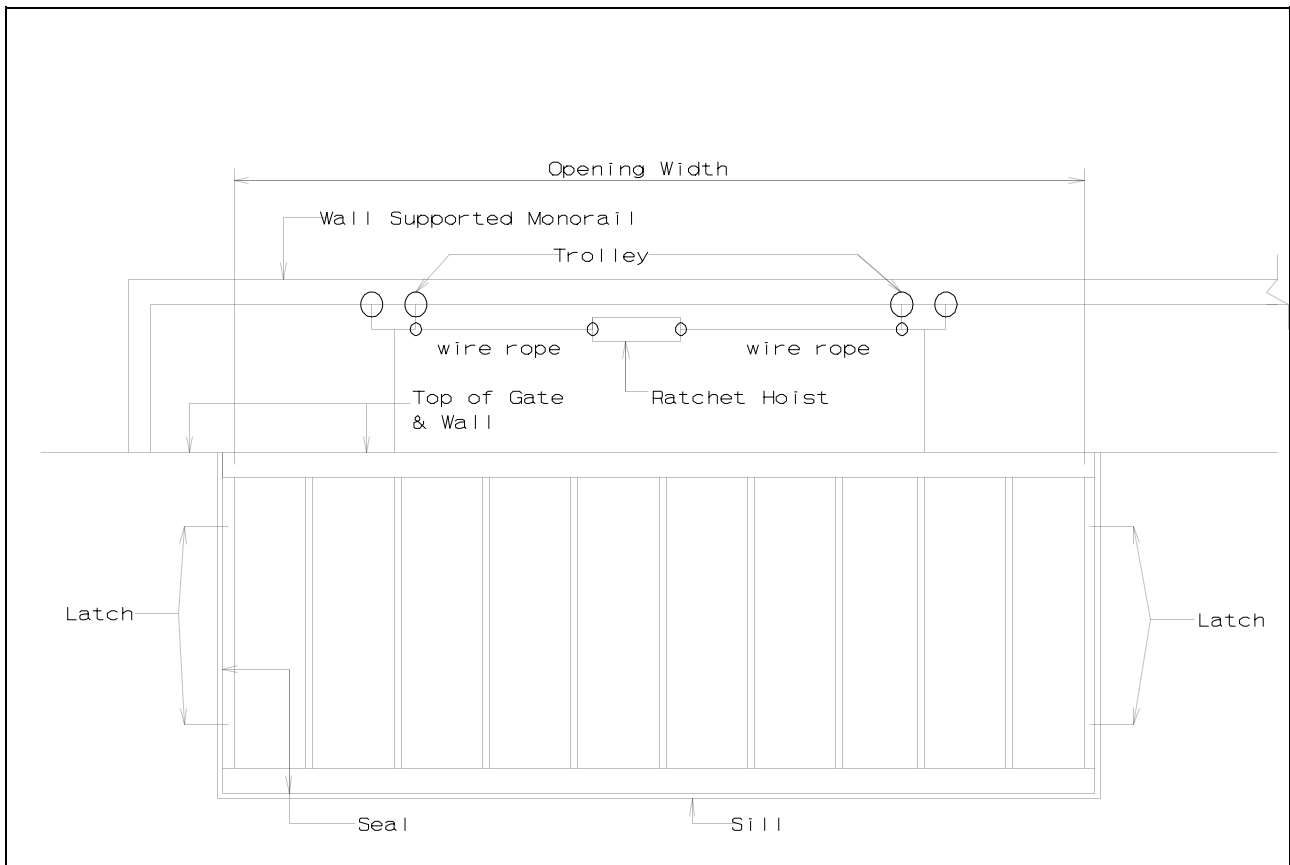

Figure 3-9. Trolley gate

Disadvantages:

- Slope of the ground adjacent to the closure opening must allow adequate clearance to open the gate.

- May be rendered inoperative due to permanent overhead support members being damaged by vehicles or other sources, or removable overhead support members or their anchorages being damaged during removal or placement operations.

- A guide member at the base of the gate may be required to support the gate against wind loads during opening and closing operations.

c. Prefabricated stoplogs and gates. Prefabricated stoplogs and gates are commercially available and are usually more cost effective for closure structures with small openings.

Chapter 4
Structural Analyses and Design

4-1. Stress Criteria

a. Concrete. Criteria for the design of reinforced concrete elements of the closure structures are given in EM 1110-2-2104.

b. Structural steel. Criteria for the design of structural steel closure gates shall be based on the provisions given in EM 1110-2-2105. Design for the normal (usual) load cases shall limit the allowable stress to 5/6 of the allowable given in AISC (1986). This "5/6" modification factor is used because Corps criteria require a higher safety factor for hydraulic structures than AISC uses for building design. Allowable stresses 1/3 greater then the normal stresses shall be used for load cases that include water to the top of the closure, wind, earthquake, or short duration loads. Load and resistance factor design criteria for structural steel closure gates are provided in EM 1110-2-2105 and AISC (1986).

c. Structural aluminum. The design of aluminum shall be based on the provisions of the Aluminum Association (1986). The allowable stress shall be limited to 5/6 of the basic values given therein for normal load cases. This allowable stress should be increased by 1/3 for design cases that include water to the top of closures, wind, earthquake, or short duration loads.

4-2. Loading Criteria

Load cases I1 through I4, given in Chapter 4 of EM 1110-2-2502 for the design of inland flood walls, shall be used for the design of closure structures. An additional load case, Case I5, is included for the design of the gate hinges and support structure for swinging gates. These load cases are described below. Load cases for closure structures loaded in coastal areas are given in Table 4-3 of EM 1110-2-2502.

a. Case I1, Design flood load. Closure in the closed position; water level on the unprotected side at the design flood level (top of wall less freeboard); water level on the unprotected side at or below the gate sill elevation.

b. Case I2, Maximum flood load. Same as Case I1 except water level is at the top of the closure on the unprotected side.

c. Case I3, Earthquake load. Water level at the usual water elevation during the nonflood stage; earthquake-induced lateral and vertical loads are acting. (Note: This case is applicable to support structures only.)

d. Case I4, Short-duration loading. Closure is subjected to short-duration loads with the gate in any position. The gate should be designed for a wind load of 15 psf during opening and closing and for a wind load as specified in EM 1110-2-2502 when the gate is in the closed position. Other special loads may be necessary for closures in hurricane flood structures to ensure safe operation during gate closures. For example, gates could be subjected to storm wave loadings during gate closing operations if gate closure is delayed as long as possible to allow for the movement of railway traffic during a storm.

e. Case I5, Gate swinging. Hinged gate of a gate closure in any position; subjected to dead load only.

4-3. Design Analysis Criteria

The design analysis criteria for closure structures are presented in detail in the design examples included in Appendices B through F. Provisions for designing and determining the stability of supporting posts, walls, and sills of closures are not included in this guidance. Chapters 4 and 5 of EM 1110-2-2502 should be referred to for overturning and sliding stability requirements. Flotation criteria are given in EM 1110-2-3104. The design of pile foundations should comply with the criteria in EM 1110-2-2906.

Chapter 5
Appurtenant Components

5-1. Gate Operating Equipment

Gate operating equipment includes motorized vehicles, winches, latches, wire rope, hooks, sheaves, snatch blocks, and other appurtenances. These items should be provided as prescribed herein and as needed for the operation of closure structures.

5-2. Seal Assemblies

Rubber seals should be of the type suitable for the particular application. Seal assemblies should be designed to fit the configuration of the gates and gate sills. Hinged bottom seal assemblies, similar to the details shown in Plate 10 are adaptable to uneven or broken sill configurations. This seal assembly is raised to clear the roadway grade during gate movements to prevent damage to the rubber seals.

5-3. Embedded Metals

Embedded structural steel bearing plates and anchorages must be provided as required for the installation of gates and appurtenances. Embedded seal plates shall be galvanized steel, stainless clad steel, or solid stainless steel for the prevention of corrosion. The edges of embedded seal plates should extend approximately 2 in. beyond the sealing surface.

Chapter 6
Corrosion Protection

6-1. Criteria

Criteria for corrosion protection of metals are given in EM 1110-2-21-5 and EM 1110-2-3400.

CLOSURE STRUCTURE
SYMMETRICAL ABOUT ℄

DRY SIDE

℄ POST & POCKET

℄ TRACK

DEFLECTOR PLATE

EL. "D"

EL. "D"
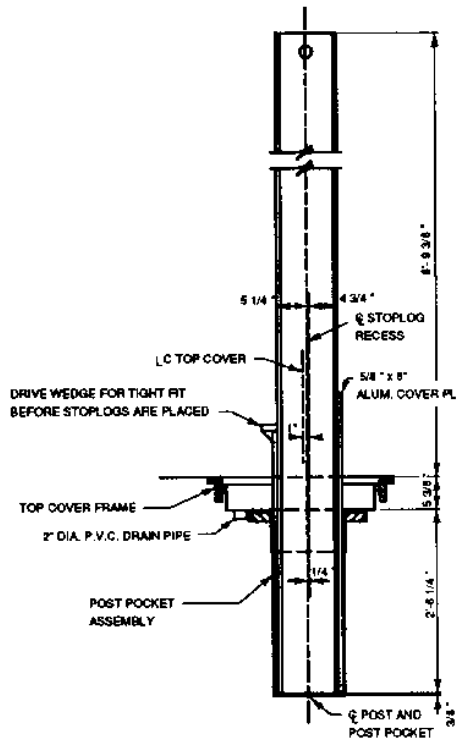
ARMOR ANGLE

WET SIDE

PLAN
SCALE: 3/8"=1'-0"

℄ TRACK

"I" WALL

℄ POST & POCKET

ALUMINUM STOPLOGS

EL. "B"

SEE
DETAIL "1"
PLATE 2C

EL. "D"

SEE
DETAIL "2", PLATE 2B

ELEVATION
SCALE: 3/8"=1'-0"

NOTE: THE DETAILS SHOWN ON THIS DRAWING WERE
USED IN A PAST CORPS OF ENGINEERS DESIGN.
THESE TYPICAL DETAILS ARE FURNISHED "FOR
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INCLUDED IN APPENDICES B-E.

GATE CLOSURES
LOCAL FLOOD PROTECTION PROJECTS

TYPICAL STOPLOG CLOSURE

DEPARTMENT OF THE ARMY
U.S. ARMY CORPS OF ENGINEERS
WASHINGTON, D.C.

PLATE 1

SECTIONAL ELEVATION AT CENTERLINE
SCALE: 1"=1'-0"

8'-9 3/8"

5 1/4" 4 3/4"

₵ STOPLOG RECESS

LC TOP COVER

5/8" x 8" ALUM. COVER PL.

DRIVE WEDGE FOR TIGHT FIT BEFORE STOPLOGS ARE PLACED

TOP COVER FRAME

2" DIA. P.V.C. DRAIN PIPE

5 3/8"

POST POCKET ASSEMBLY

1/4"

2'-8 1/4"

₵ POST AND POST POCKET

3/4"

POST DETAIL
SCALE: 1"=1'-0"

NOTE:
WALL RECESS AND POST FLANGE BEARING FACES SHALL LIE IN THE SAME THEORETICAL VERTICAL PLANE.

DRILL 24/16"⌀ HOLES FOR 1/2"⌀ BOLTS (COUNTERSUNK FAR SIDE)

11'-0"

5" 5"

SPECIAL ALUM. SECTION (ALUM. ALLOY 7075-T6 FURNISHED BY OTHERS)

RIVETS HEADS MUST BE FLUSH WITH BEARING PLANE

10 5/8" EXACT

4'-0"

3'-3"

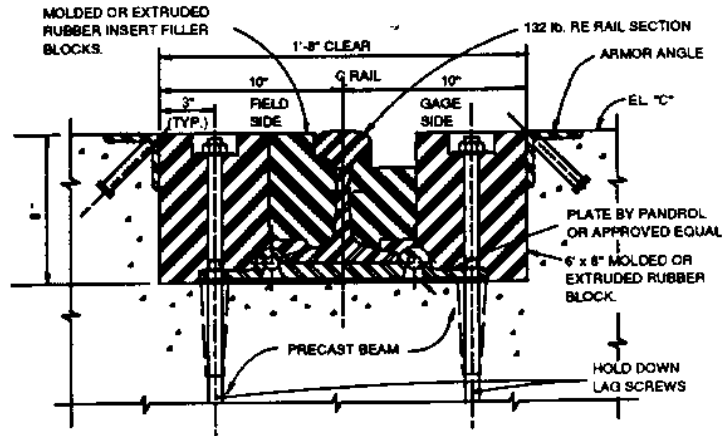## SECTION
SCALE: 1 1/2"=1'-0"

10"

5/8" R

1/2" R

1/2" R

3/4"

5/8"

GATE CLOSURES
LOCAL FLOOD PROTECTION PROJECTS
TYPICAL STOPLOG CLOSURE DETAILS I

DEPARTMENT OF THE ARMY
U.S. ARMY CORPS OF ENGINEERS
WASHINGTON, D.C.

PLATE 2A

SECTION A

SCALE: 1"=1'-0"

EMBEDDED POCKET WITH POST IN PLACE

SCALE: 1"=1'-0"

Labels (Section): STOPLOG RECESS AND SILL BEAM; ℄ POST POCKET; 2" DIA. PVC DRAIN PIPE; TOP COVER FRAME; WEDGE; POST POCKET ASSY.; REMOVABLE POST (IN PLACE); 1'-4"; 1'-2"; 1/4"

DETAIL 2

SCALE: 1"=1'-0"

Labels (Detail 2): CAST IRON COVER PLATE NOT SHOWN; 1/2 ⌀ BENT ROD (HANDHOLD); TYP.; 1 1/2" TYP.; COVER PLATE SHOWN IN PLACE; DRAIN PIPE NOT SHOWN; TOP OF SILL; BAR 1 1/2"x3 1/2" CONTINUOUS; DRILL & TAP FOR ⌀1/2"/x2" LG. HEX. HD. BOLT & WASHER, USE POCKET COVER P FOR HOLE TEMPLATE, (TYP. FOR BOTH POST POCKET ASSEMBLY); L-7"x4"x 1/2" x 1'-4 1/4" 2-REQUIRED EACH POST POCKET ASSEMBLY; LAND SIDE; 11" (EXACT); 5 1/2"-5 1/2"; 3/8"R WELDED TOGETHER AT CORNERS; ℄ POST & POST POCKET; TYP.

WEDGE DETAIL

SCALE: 1 1/2"=1'-0"

Labels (Wedge Detail): BAR 1"x2 1/2"x0'-8"; 2 1/2"; 1 1/2"; 1"R; 5/8" EXACT; 9/16" SLOTTED HOLES FOR 1/2"⌀ SLOTTED HEAD COUNTER SUNK BOLTS WITH HEX. NUTS AND WASHERS, GALVANIZED.; STRUCTURAL STEEL (GALVANIZED); P 8"x 5/8"x1'-8" (TAPER AS SHOWN); 3/8" EXACT

CLAMP ANCHOR ASSEMBLY

SCALE: NONE

Labels (Clamp Anchor Assembly): CLAMP; ℄ BULKHEAD RECESS; GATE CLOSED; GATE OPEN; CLAMP ROD; SYM ABT ℄; ALUMINUM STOP LOGS; SILL BEAM; PLUG BOLT
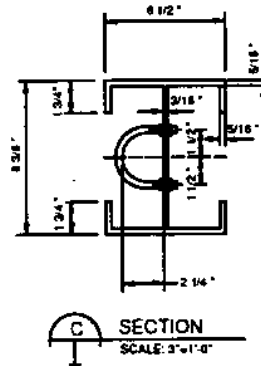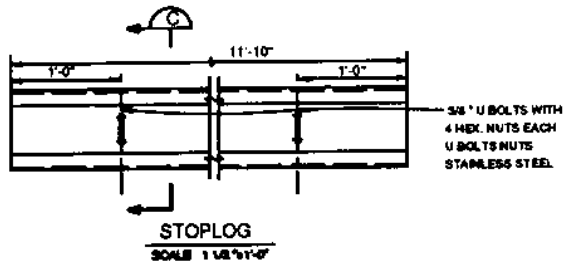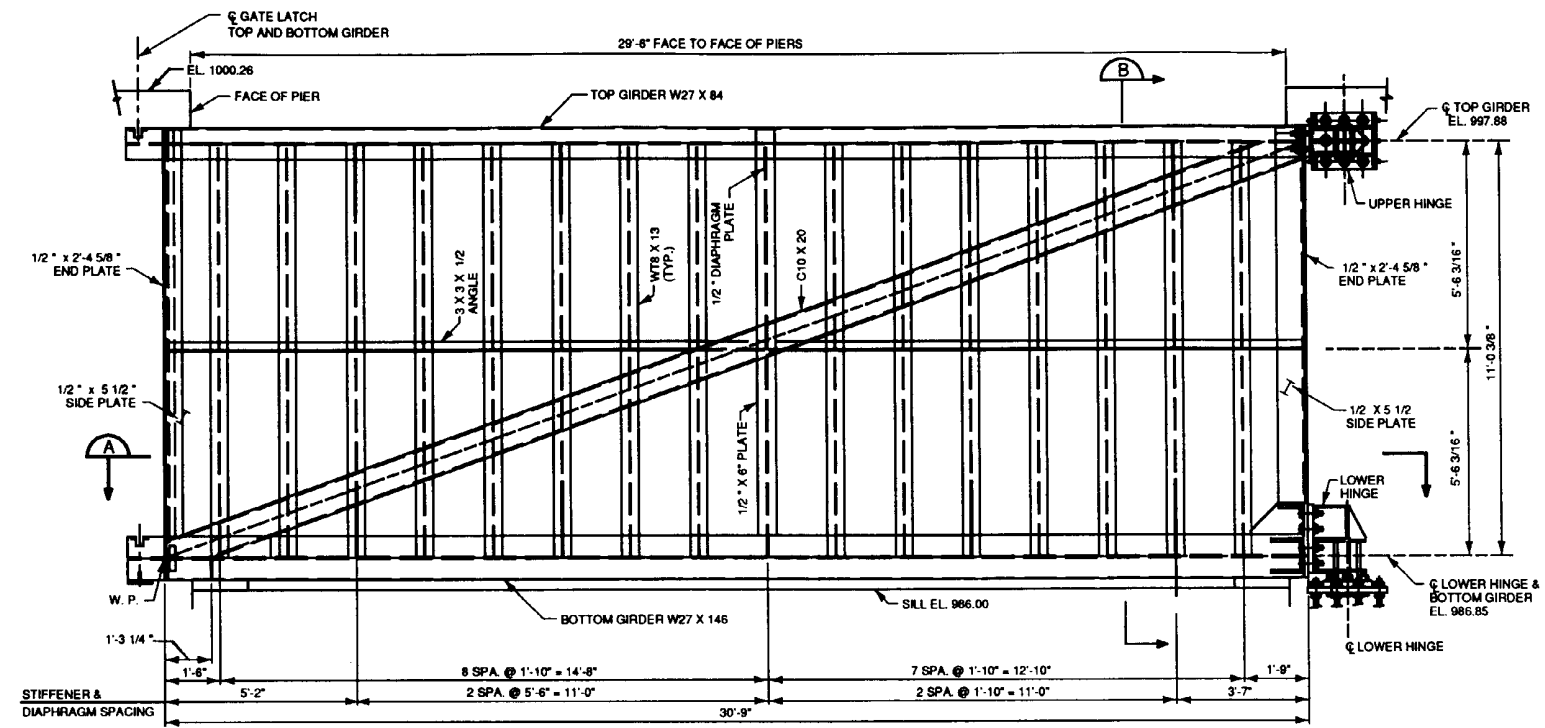
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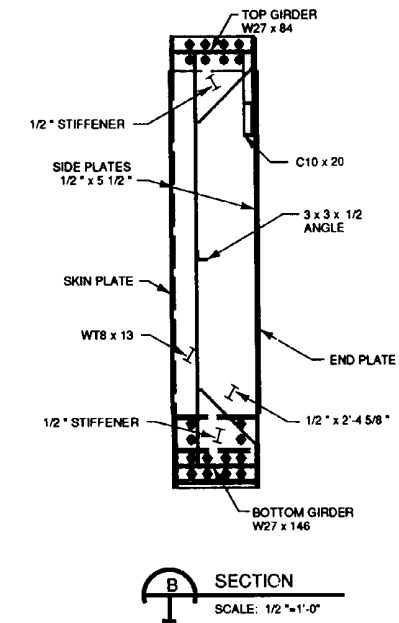
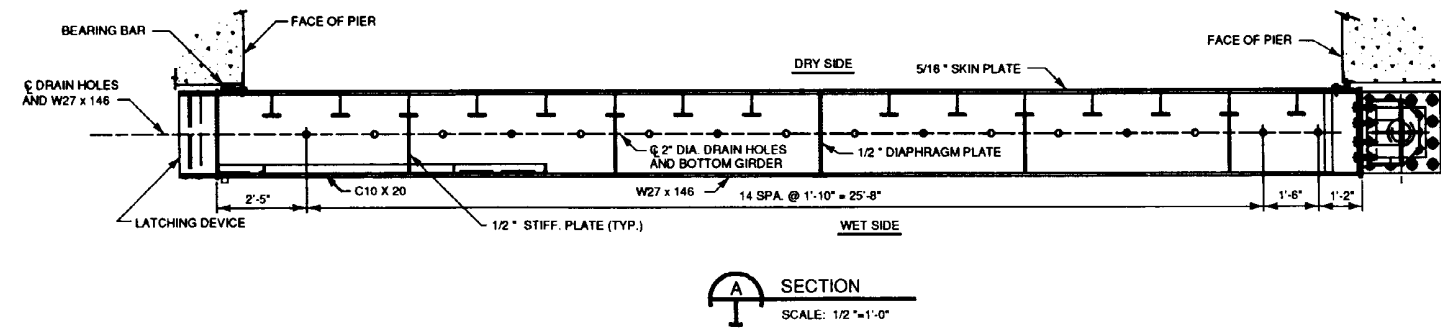| GATE CLOSURES |
| --- |
| LOCAL FLOOD PROTECTION PROJECTS |
| TYPICAL STOPLOG CLOSURE DETAILS II |
| DEPARTMENT OF THE ARMY U.S. ARMY CORPS OF ENGINEERS WASHINGTON, D.C. |
| PLATE 28 |

MOLDED OR EXTRUDED
RUBBER INSERT FILLER
BLOCKS.

132 lb. RE RAIL SECTION

1'-8" CLEAR

ARMOR ANGLE

10"

₵ RAIL

10"

3"
(TYP.)

FIELD
SIDE

GAGE
SIDE

EL "C"

PLATE BY PANDROL
OR APPROVED EQUAL

6" x 6" MOLDED OR
EXTRUDED RUBBER
BLOCK.

PRECAST BEAM

HOLD DOWN
LAG SCREWS

DETAIL 1
RAIL ASSEMBLY
SCALE: 3"=1'-0"

11'-10"

1'-0"

1'-0"

3/8 " U BOLTS WITH
4 HEX. NUTS EACH
U BOLTS NUTS
STAINLESS STEEL

STOPLOG
SCALE: 1 1/2"=1'-0"

6 1/2 "

5/16"

3/4"

3/16"

5/16"

8 3/8"

1 1/2"

3/4"

2 1/4 "

C SECTION
SCALE: 3"=1'-0"

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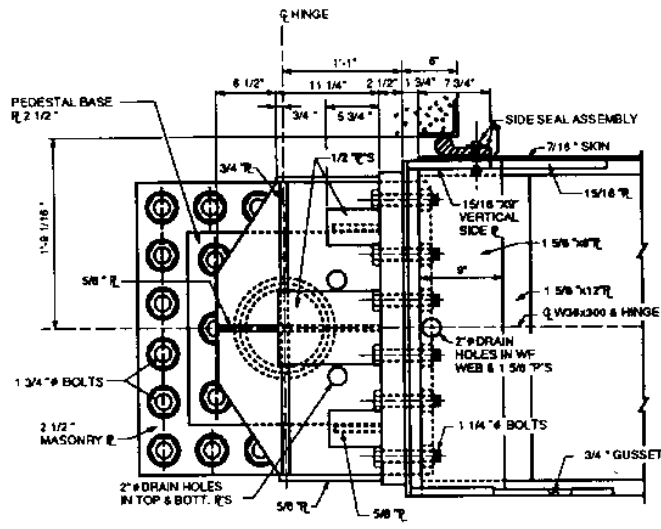## WET SIDE ELEVATION
SCALE: 1/2 "=1'-0"

SECTION A
SCALE: 1/2 "=1'-0"

SECTION B
SCALE: 1/2 "=1'-0"

NOTE: THE DETAILS SHOWN ON THIS DRAWING WERE USED IN A PAST CORPS OF ENGINEERS DESIGN. THESE TYPICAL DETAILS ARE FURNISHED "FOR INFORMATION ONLY" AND MAY NOT CORRESPOND WITH THE CRITERIA AND DESIGN EXAMPLES INCLUDED IN APPENDICES B THROUGH E.
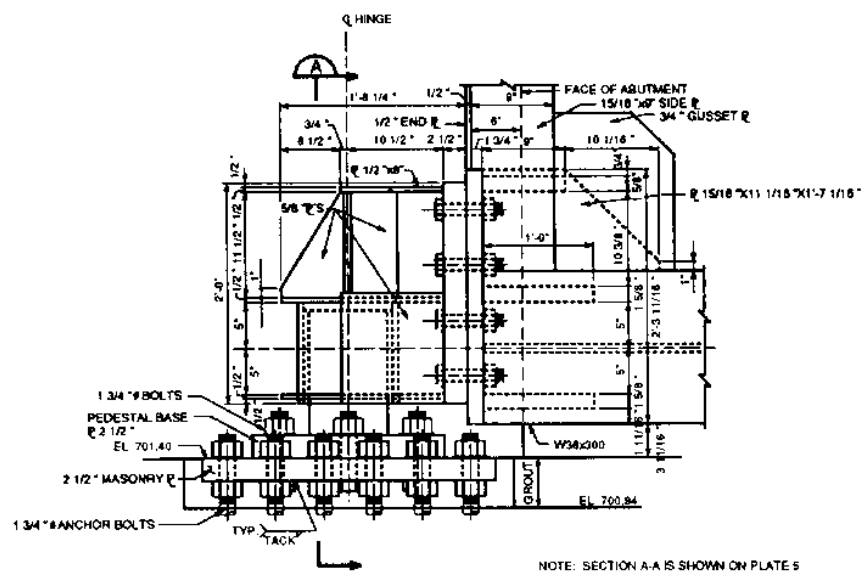
NOTE:
1. TYPICAL HINGE DETAILS ARE SHOWN ON PLATES 4, 5, 6, & 7.
2. TYPICAL SEAL DETAILS ARE SHOWN ON PLATES 8, 9, & 10.
3. TYPICAL LATCH DETAILS ARE SHOWN ON PLATES 11 & 12.

GATE CLOSURES
LOCAL FLOOD PROTECTION PROJECTS

TYPICAL SWING GATE

DEPARTMENT OF THE ARMY
U.S. ARMY CORPS OF ENGINEERS
WASHINGTON, D.C.

PLATE 3

PLAN
SCALE: 1 1/2"=1'-0"

ELEVATION
SCALE: 1 1/2"=1'-0"

NOTE: SECTION A-A IS SHOWN ON PLATE 5
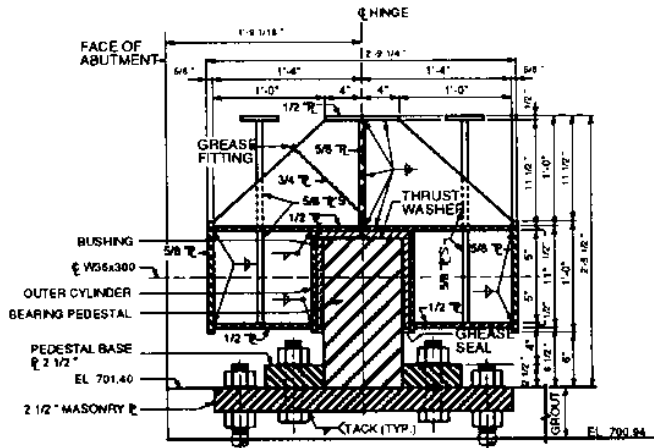
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GATE CLOSURES
LOCAL FLOOD PROTECTION PROJECTS

TYPICAL LOWER HINGE DETAILS I

DEPARTMENT OF THE ARMY
U.S. ARMY CORPS OF ENGINEERS
WASHINGTON, D.C.

PLATE 4

REQUIRED LIST

MARK	NO. REQ.	DESCRIPTION	MATERIAL	EST. WT (LBS. EA.)	USED WITH	REMARKS
13/1	1	BUSHING	BRONZE		13/13	ASTM B22 UNS NO. C91300
13/2	1	HINGE PIN	STAINLESS STEEL			ASTM A276 TYPE XM19
13/3	1	RECESSED PIN NUT	CRES		13/2	
13/4	1	THRUST WASHER	BRONZE		13/6	ASTM B22 UNS NO. C91300
13/5	1	BUSHING	BRONZE		13/14	ASTM B22 UNS NO. C91300
13/6	1	BEARING PEDESTAL	STAINLESS STEEL	158	13/7	ASTM A276 TYPE XM19
13/7	1	PEDESTAL BASE PLATE	STAINLESS STEEL	271	13/6	ASTM A240
13/8	20	ANCHOR BOLT	SEE NOTE A		13/8	
13/9	20	ANCHOR PLATE	STEEL	70		ASTM A-36
13/10	AS REQ'D	SHIM PLATE	STEEL		TOP HINGE	ASTM A-36
13/11	10	ANCHOR PLATE		407	TOP HINGE	
13/12	10	ANCHOR PLATE	STEEL	35	13/11	ASTM A-36
13/13	1	OUTER CYLINDER	STEEL	26	13/1	ASTM A-53 GRADE B OR
13/14	1	OUTER CYLINDER	STEEL		13/5	ASTM A-501
13/A	8	1 3/4 "⌀X7 3/8 " HEX HD BOLT (3 1/4 "THD LG) W/ HEAVY HEX NUT & WASHER	SEE NOTE A		BOTT. HINGE	
13/B	1	GREASE SEAL "HDS1 NO. 503007" AS MFR. BY C.R.IND. OR APP. EQUAL 10.0" O.D.				COMMERCIAL GRADE
13/C	2	GREASE FITTINGS 1/8 " N PT TYPE				COMMERCIAL GRADE
13/D	20	1 1/4 "⌀X6 1/4 " HEX HD BOLT (2 3/4 "THD LG) W/ HEAVY HEX NUT & WASHER	SEE NOTE A		BOTT. HINGE	
13/E	8	SET SCREW—HEX SOCKET 1 1/4 "X3 3/8 " CLASS #3 W/HEX JAM NUTS	STEEL		TOP HINGE	ASTM A-325
13/F	2	GREASE SEAL "C.R.IND. TYPE CRWH1 #39933"-3.99" I.D. OR APP. EQUAL				COMMERCIAL GRADE
13/G	10	1 1/2 "⌀X5 1/4 " (2 3/4 "THD LG) W/ HEAVY HEX NUT & FLAT WASHER	SEE NOTE A		TOP HINGE	

NOTE "A": A.S.T.M. A-193, CLASS 1-C, GRADE B8R OR B8RA BOLTS
WITH ASTM A194 TYPE 303, GR.8FA HEAVY HEX. NUTS.

NOTE: QUANTITIES SHOWN ARE FOR ONE GATE ONLY.

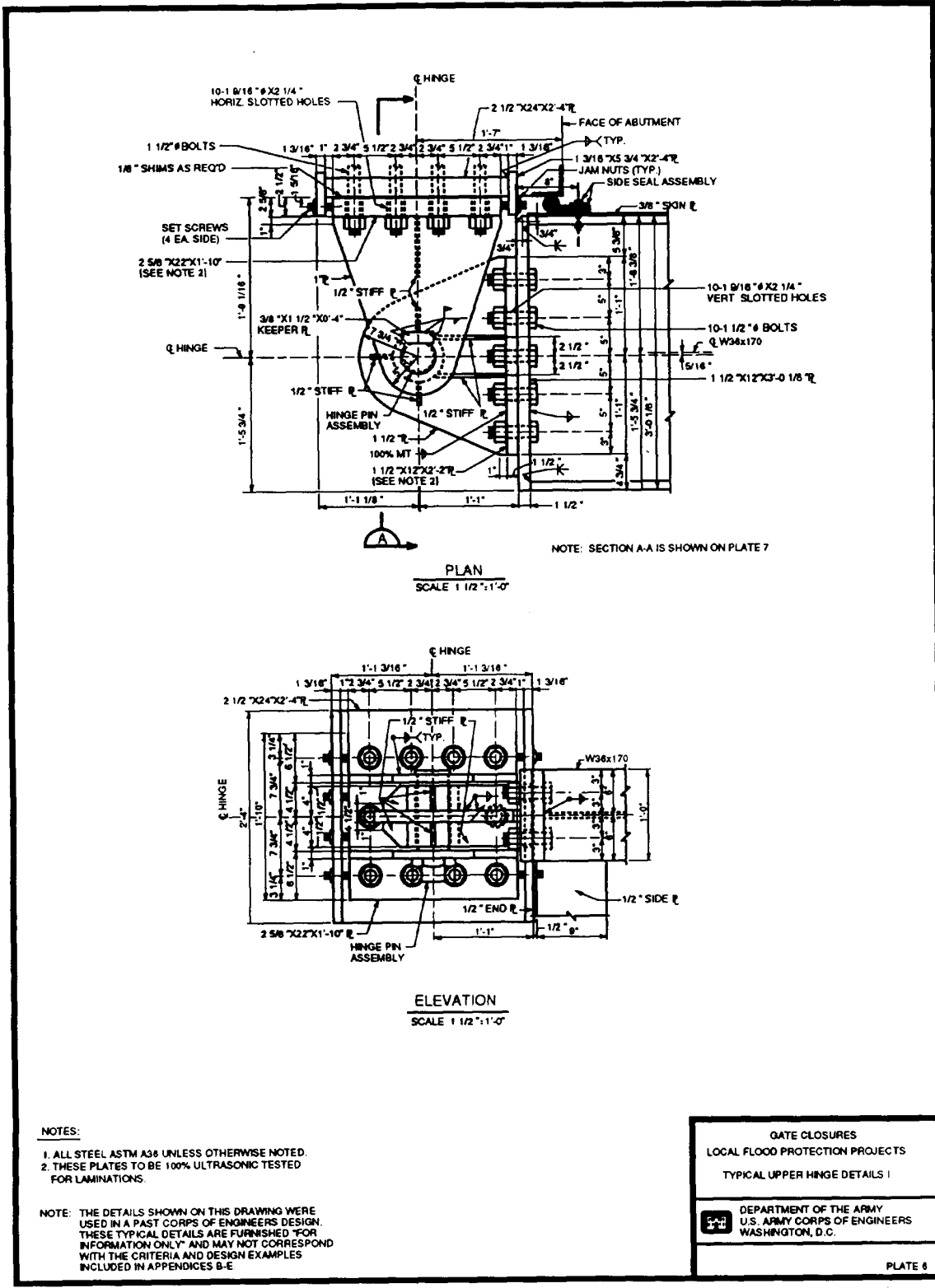NOTE: 1. ALL STEEL ASTM A36, UNLESS OTHERWISE NOTED.

SECTION
SCALE 1 1/2 "=1'-0"

GATE CLOSURES
LOCAL FLOOD PROTECTION PROJECTS

TYPICAL LOWER HINGE DETAILS II

DEPARTMENT OF THE ARMY
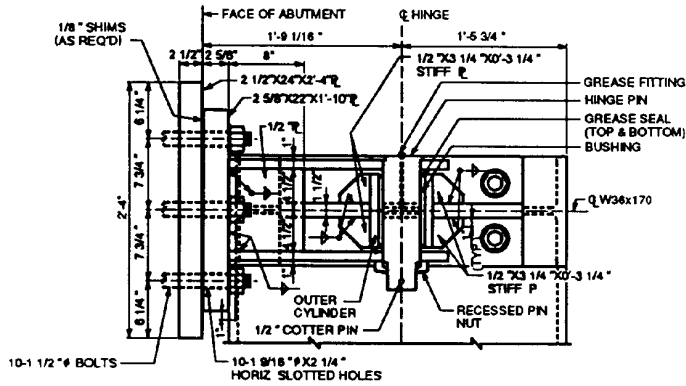U.S. ARMY CORPS OF ENGINEERS
WASHINGTON, D.C.

PLATE 5

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INCLUDED IN APPENDICES B-E

PLAN
SCALE 1 1/2":1'-0"

NOTE: SECTION A-A IS SHOWN ON PLATE 7

ELEVATION
SCALE 1 1/2":1'-0"

NOTES:

1. ALL STEEL ASTM A36 UNLESS OTHERWISE NOTED.
2. THESE PLATES TO BE 100% ULTRASONIC TESTED
   FOR LAMINATIONS.

NOTE: THE DETAILS SHOWN ON THIS DRAWING WERE
USED IN A PAST CORPS OF ENGINEERS DESIGN.
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INCLUDED IN APPENDICES B-E.

GATE CLOSURES
LOCAL FLOOD PROTECTION PROJECTS
TYPICAL UPPER HINGE DETAILS I

DEPARTMENT OF THE ARMY
U.S. ARMY CORPS OF ENGINEERS
WASHINGTON, D.C.

PLATE 6

SECTION
SCALE 1 1/2 "=1'-0"

GATE CLOSURES
LOCAL FLOOD PROTECTION PROJECTS

TYPICAL UPPER HINGE DETAILS II

DEPARTMENT OF THE ARMY
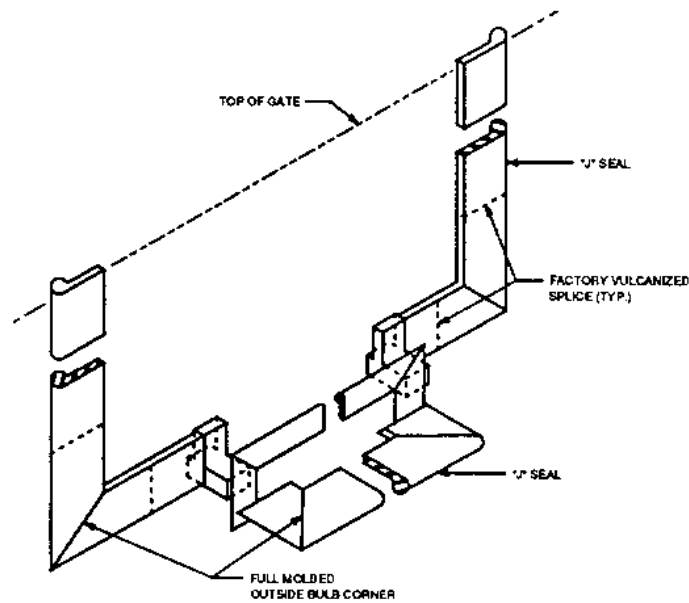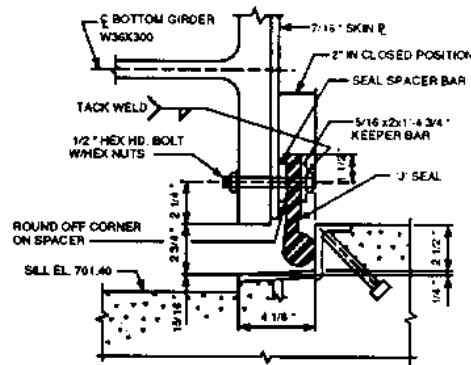U.S. ARMY CORPS OF ENGINEERS
WASHINGTON, D.C.

PLATE 7

LAYOUT OF GATE SEAL
N.T.S.

TYPICAL FIXED BOTTOM SEAL
SCALE: 3"=1'-0"

NOTES:
1. ALL STEEL ASTM A36, UNLESS OTHERWISE NOTED
2. ALL SEAL BOLTS AND NUTS SHALL BE CRES.
3. "U" SEALS TO HAVE 50 TO 60 DUROMETER HARDNESS.

NOTE: THE DETAILS SHOWN ON THIS DRAWING WERE
USED IN A PAST CORPS OF ENGINEERS DESIGN.
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GATE CLOSURES
LOCAL FLOOD PROTECTION PROJECTS

TYPICAL SEAL DETAILS I

DEPARTMENT OF THE ARMY
U.S. ARMY CORPS OF ENGINEERS
WASHINGTON, D.C.

PLATE 6

LATCH END
SCALE: 6"=1'

HINGE END
SCALE: 6"=1'
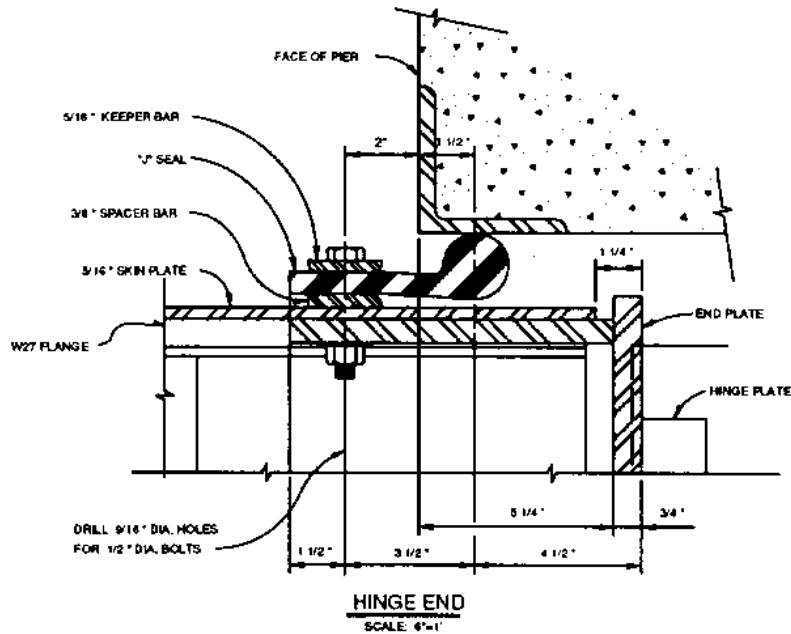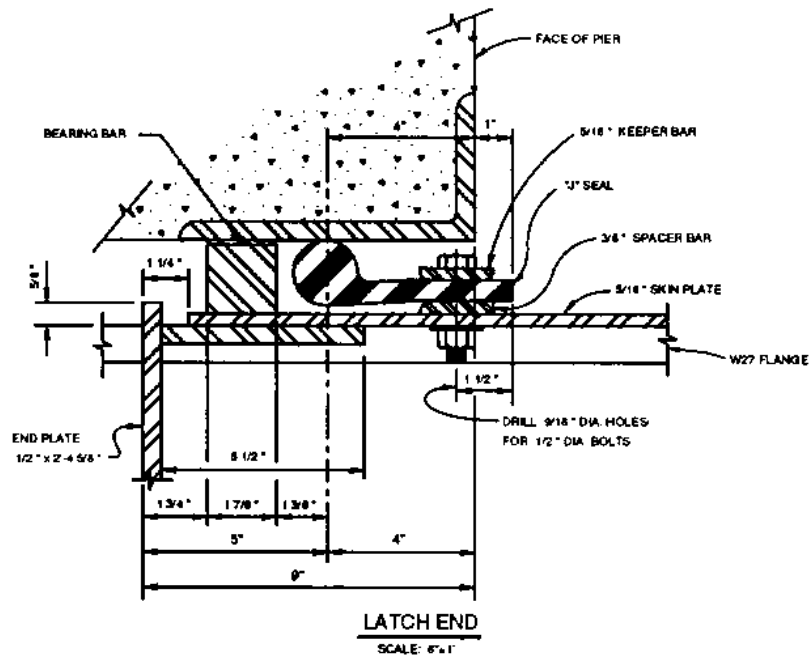
TYPICAL VERTICAL SIDE SEAL DETAILS

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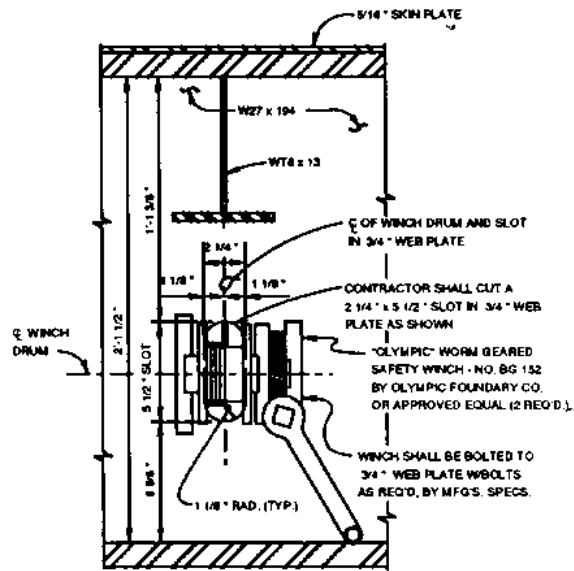INCLUDED IN APPENDICES B-E.

GATE CLOSURES
LOCAL FLOOD PROTECTION PROJECTS

TYPICAL SEAL DETAILS II

DEPARTMENT OF THE ARMY
U.S. ARMY CORPS OF ENGINEERS
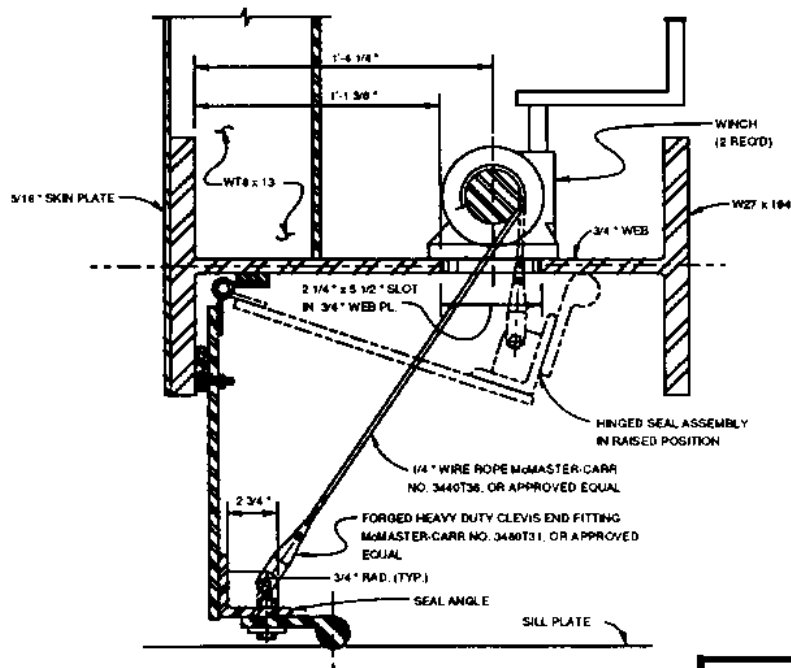WASHINGTON, D.C.

PLATE 9

5/16" SKIN PLATE

W27 x 194

WT8 x 13

1'-1 3/8"

2 1/4"

1 1/8" | 1 1/8"

1 1/8"

2'-1 1/2"

5 1/2" SLOT

6 3/4"

₵ WINCH
DRUM

₵ OF WINCH DRUM AND SLOT
IN 3/4" WEB PLATE

CONTRACTOR SHALL CUT A
2 1/4" x 5 1/2" SLOT IN 3/4" WEB
PLATE AS SHOWN

"OLYMPIC" WORM GEARED
SAFETY WINCH - NO. BG 152
BY OLYMPIC FOUNDRY CO.
OR APPROVED EQUAL (2 REQ'D.).

WINCH SHALL BE BOLTED TO
3/4" WEB PLATE W/BOLTS
AS REQ'D. BY MFG'S. SPECS.

1 1/8" RAD. (TYP.)

PLAN
SCALE: 3"=1'-0"

5/16" SKIN PLATE

WT8 x 13

1'-4 1/4"

1'-1 3/8"

WINCH
(2 REQ'D)

W27 x 194

3/4" WEB

2 1/4" x 5 1/2" SLOT
IN 3/4" WEB PL.

HINGED SEAL ASSEMBLY
IN RAISED POSITION

1/4" WIRE ROPE McMASTER-CARR
NO. 3440T36, OR APPROVED EQUAL

2 3/4"

FORGED HEAVY DUTY CLEVIS END FITTING
McMASTER-CARR NO. 3460T31, OR APPROVED
EQUAL

3/4" RAD. (TYP.)

SEAL ANGLE

SILL PLATE

TYPICAL RETRACTABLE BOTTOM SEAL
SCALE: 3"=1'-0"

NOTE: THE DETAILS SHOWN ON THIS DRAWING WERE
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GATE CLOSURES
LOCAL FLOOD PROTECTION PROJECTS

TYPICAL SEAL DETAILS III

DEPARTMENT OF THE ARMY
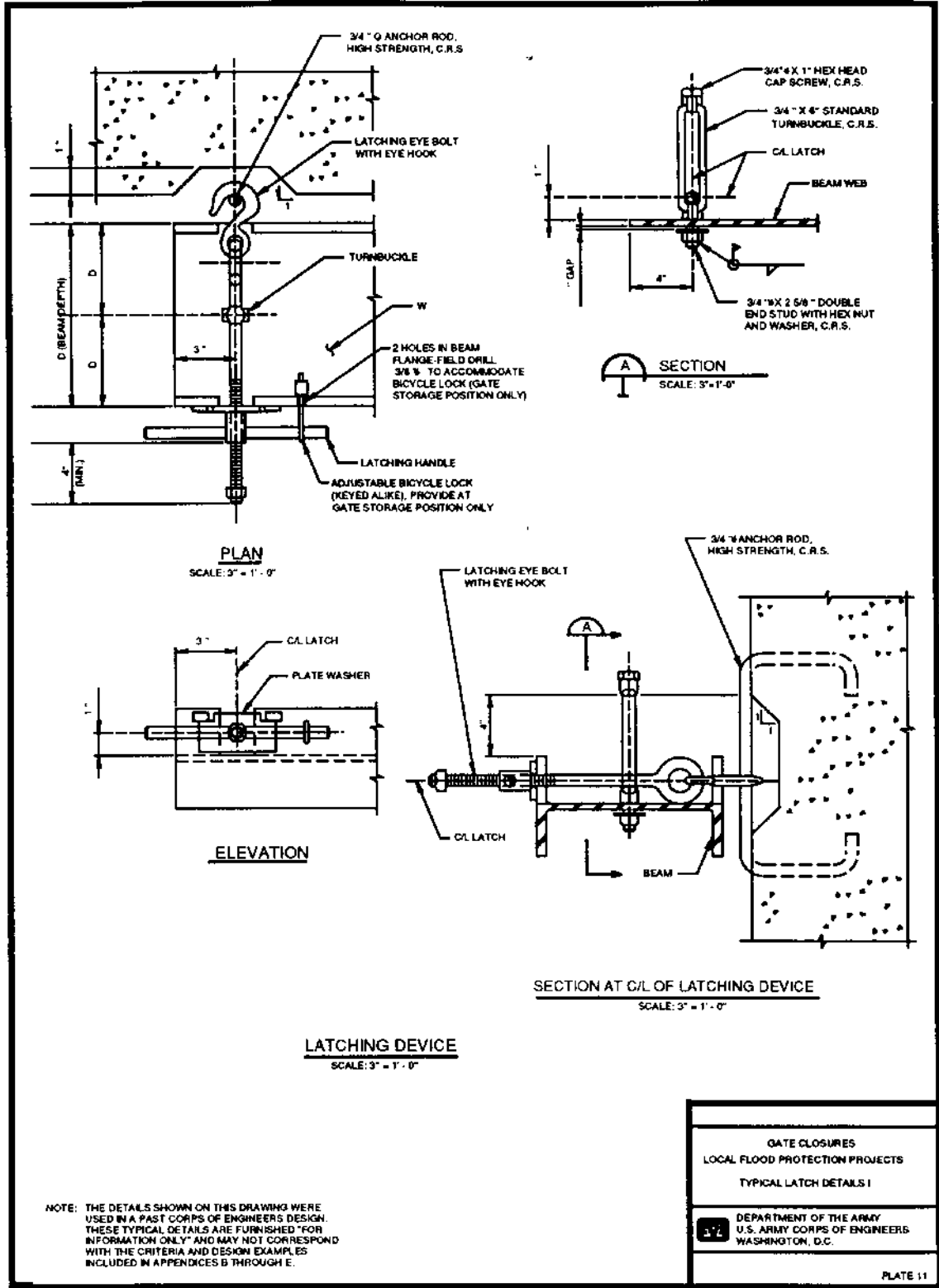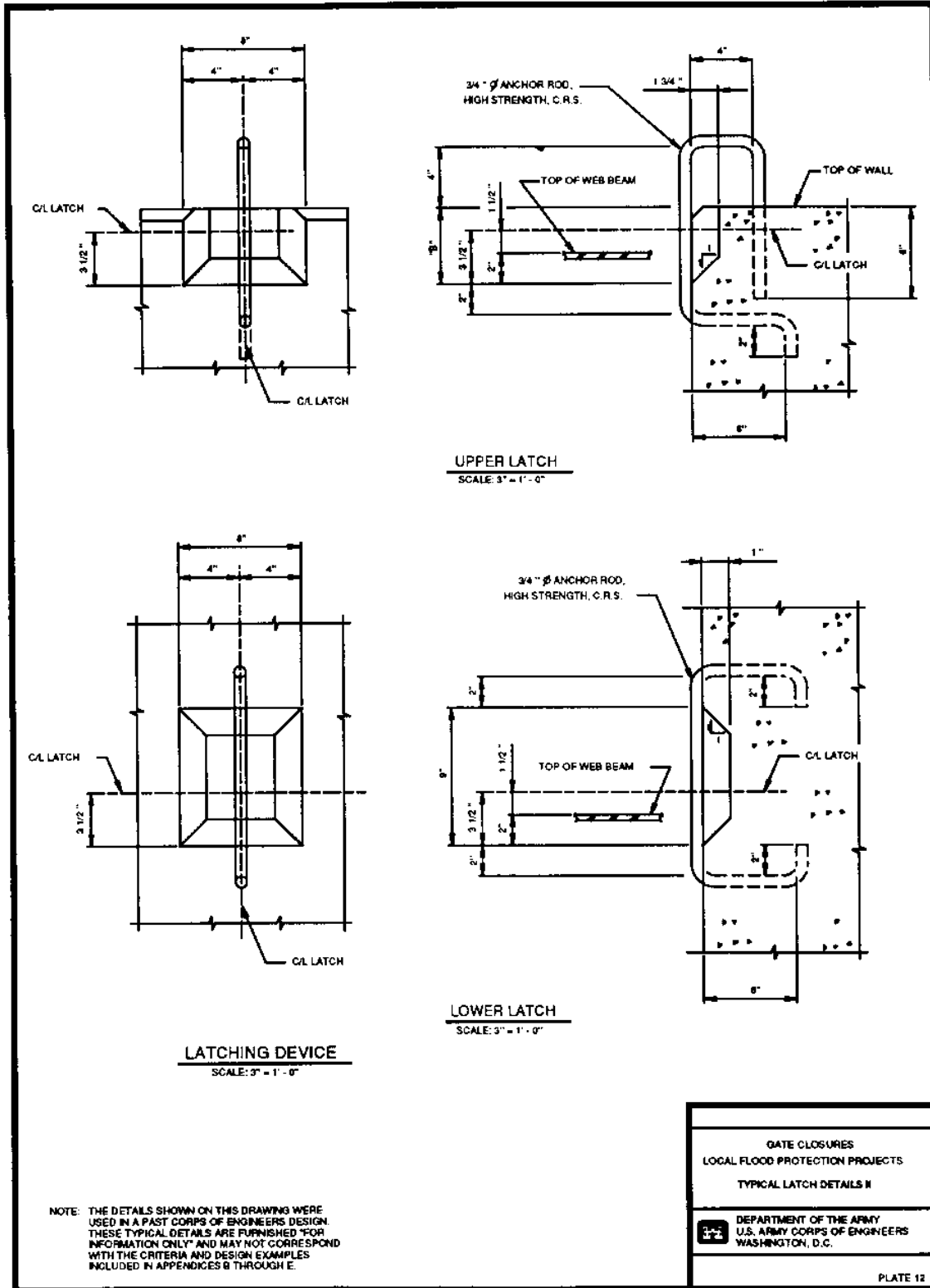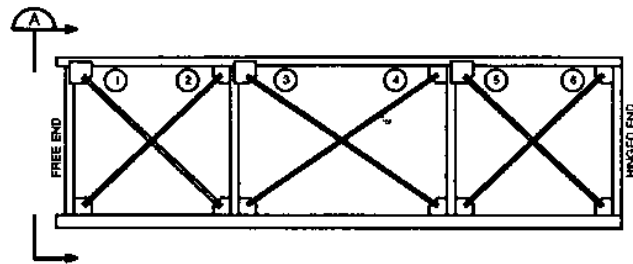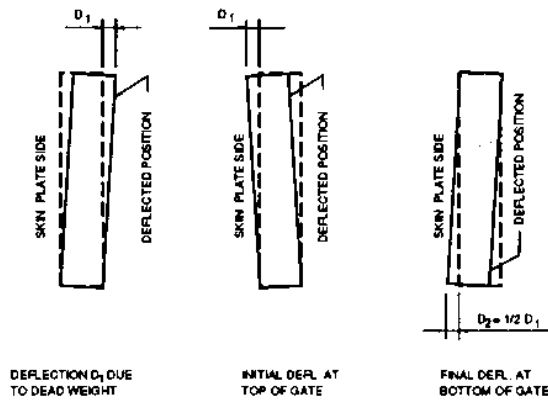U.S. ARMY CORPS OF ENGINEERS
WASHINGTON, D.C.

PLATE 10

3/4 " Q ANCHOR ROD,
HIGH STRENGTH, C.R.S

LATCHING EYE BOLT
WITH EYE HOOK

TURNBUCKLE

W

D (BEAM DEPTH)

D

3"

D

2 HOLES IN BEAM
FLANGE-FIELD DRILL
3/8 % TO ACCOMMODATE
BICYCLE LOCK (GATE
STORAGE POSITION ONLY)

4" (MIN.)

LATCHING HANDLE

ADJUSTABLE BICYCLE LOCK
(KEYED ALIKE), PROVIDE AT
GATE STORAGE POSITION ONLY

PLAN
SCALE: 3" = 1' - 0"

3/4" x 1" HEX HEAD
CAP SCREW, C.R.S.

3/4 " X 4" STANDARD
TURNBUCKLE, C.R.S.

C/L LATCH

BEAM WEB

GAP

4"

3/4 "% X 2 5/8 " DOUBLE
END STUD WITH HEX NUT
AND WASHER, C.R.S.

A SECTION
SCALE: 3" = 1'-0"

3"

C/L LATCH

PLATE WASHER

1"

ELEVATION

LATCHING EYE BOLT
WITH EYE HOOK

A

3/4 "% ANCHOR ROD,
HIGH STRENGTH, C.R.S.

C/L LATCH

BEAM

SECTION AT C/L OF LATCHING DEVICE
SCALE: 3" = 1' - 0"

LATCHING DEVICE
SCALE: 3" = 1' - 0"

GATE CLOSURES
LOCAL FLOOD PROTECTION PROJECTS

TYPICAL LATCH DETAILS I

DEPARTMENT OF THE ARMY
U.S. ARMY CORPS OF ENGINEERS
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PLATE 11

NOTE: THE DETAILS SHOWN ON THIS DRAWING WERE
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UPPER LATCH
SCALE: 3" = 1' - 0"

3/4 " Ø ANCHOR ROD,
HIGH STRENGTH, C.R.S.

TOP OF WEB BEAM

TOP OF WALL

C/L LATCH

C/L LATCH

C/L LATCH

LOWER LATCH
SCALE: 3" = 1' - 0"

3/4 " Ø ANCHOR ROD,
HIGH STRENGTH, C.R.S.

TOP OF WEB BEAM

C/L LATCH

C/L LATCH

LATCHING DEVICE
SCALE: 3" = 1' - 0"

C/L LATCH

C/L LATCH

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GATE CLOSURES
LOCAL FLOOD PROTECTION PROJECTS

TYPICAL LATCH DETAILS II

DEPARTMENT OF THE ARMY
U.S. ARMY CORPS OF ENGINEERS
WASHINGTON, D.C.

PLATE 12

DIAGONAL BRACE DIAGRAM (SIM.)
SCALE: NONE

DEFLECTION D_1 DUE
TO DEAD WEIGHT

INITIAL DEFL. AT
TOP OF GATE

FINAL DEFL. AT
BOTTOM OF GATE

$D_2 = 1/2\ D_1$

END SECTION
SCALE: NONE

NOTES:

1. A DEFLECTION OF THE GATE IS DEFINED AS A TWISTING OF THE GATE SUCH THAT THE FREE END IS OUT OF PLUMB. A POSITIVE DEFLECTION OF THE LEAF IS ONE IN WHICH THE TOP OF THE FREE END IS MOVED AWAY FROM THE SKINPLATE SIDE RELATIVE TO THE BOTTOM. THE MAGNITUDE OF THE DEFLECTION IS THE AMOUNT BY WHICH THE FREE END IS OUT OF PLUMB, AS SHOWN IN THE SKETCH AS D_1

2. WHEN ANY DIAGONALS ARE TIGHTENED, THEY SHALL BE TAKEN UP JUST TO THE POINT WHERE ALL OF THE SLACK IS REMOVED AND A VERY SLIGHT TENSION EXISTS. CARE SHALL BE EXERCISED TO SEE THAT THE AMOUNT OF THIS INITIAL TENSION IS AS SMALL AS POSSIBLE. THE SLACK SHALL BE CONSIDERED TO BE REMOVED WHEN THE DIAGONAL DOES NOT BOW IN OR OUT FROM THE GATE. NO ATTEMPT SHALL BE MADE TO REMOVE THE SLIGHT VERTICAL SAG WHICH WILL ALWAYS EXIST IN THE DIAGONAL BECAUSE OF ITS DEAD WEIGHT.

3. ALL STEEL ASTM A-36 UNLESS NOTED OTHERWISE.

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USED IN A PAST CORPS OF ENGINEERS DESIGN.
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GATE CLOSURES
LOCAL FLOOD PROTECTION PROJECTS
TYPICAL DIAGONAL PRESTRESSING
NOTES I

DEPARTMENT OF THE ARMY
U.S. ARMY CORPS OF ENGINEERS
WASHINGTON, D.C.

PLATE 13A

METHOD OF PRESTRESSING DIAGONALS

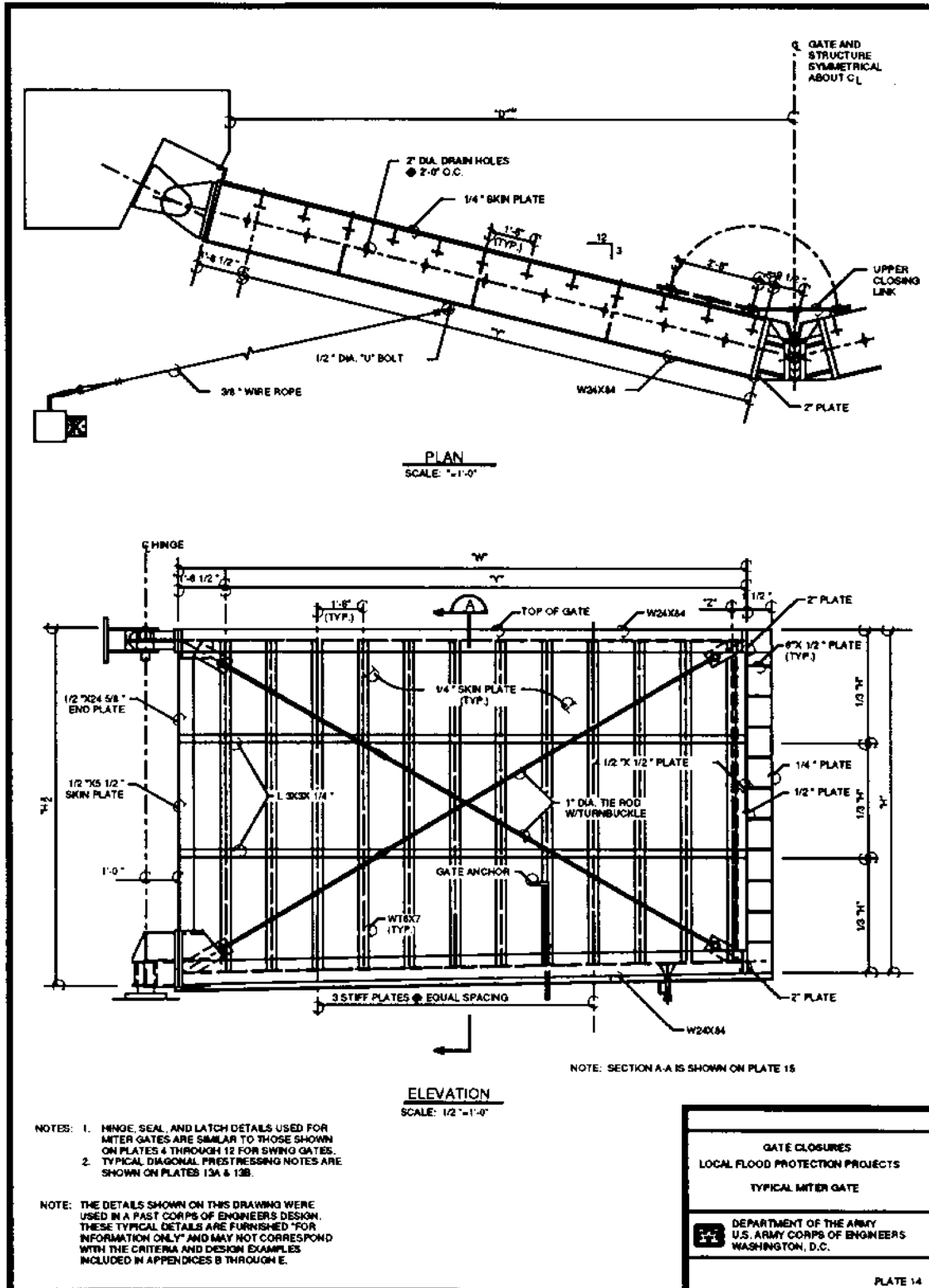1. ENSURE THAT ALL DIAGONALS ARE SLACK.

2. LUBRICATE THE TURNBUCKLES ON THE DIAGONALS SO THEY CAN
   TURN EASILY.

3. ROSETTES FOR THE STRAIN GAGES SHALL BE PLACED ON ALL
   DIAGONALS A MINIMUM OF 20 HR BEFORE PRESTRESSING, UNLESS
   AN APPROVED QUICK-SETTING CEMENT IS USED.

4. WITHOUT THE RESTRAINT OF ANY GUYS OR JACKS, THE GATE WILL
   DEFLECT IN A POSITIVE DIRECTION UNDER ITS OWN DEAD WEIGHT.
   THIS DEFLECTION (D_1) SHALL BE MEASURED.

5. THE LEAF SHALL BE GUYED AT THE BOTTOM OF ITS FREE END,
   AND A JACKING OR PULLING DEVICE SHALL BE PLACED AT THE
   TOP OF THE FREE END.

6. MOVE THE FREE END TOWARD THE SKINPLATE SIDE UNTIL THE LEAF
   HAS A DEFLECTION IN A NEGATIVE DIRECTION EQUAL TO D_1 AT
   THE TOP.

7. HOLD THIS DEFLECTION AND TIGHTEN ALL OF THE DIAGONALS. THESE
   DIAGONALS SHALL BE TIGHTENED SO THAT THERE IS NO HORIZONTAL
   BOW. NO ATTEMPT WILL BE MADE TO REMOVE ALL OF THE
   VERTICAL SAG.

8. RESTRAIN THE TOP OF THE FREE END LEAF WITH GUYS.

9. PROCEED WITH THE JACKING AT THE BOTTOM OF THE FREE END
   UNTIL A DEFLECTION D_2 IS OBTAINED. DURING THIS OPERATION,
   THE ADJUSTMENT OF DIAGONALS 1, 3, AND 5 WILL NOT BE CHANGED.
   HOWEVER, TIGHTENING OF DIAGONALS 2, 4, AND 6 WILL BE CONTINUED
   UNTIL THERE IS A SLIGHT TENSION IN THE MEMBERS WHEN THE GATE
   IS IN ITS FINAL DEFLECTED POSITION.

10. DURING THE PRESTRESSING OPERATION, A STRAIN GAGE WILL BE
    USED TO DETERMINE THE STRESS IN THE DIAGONALS. THE MAXIMUM
    ALLOWABLE STRESS IN THE DIAGONALS DURING PRESTRESSING
    SHALL BE 20,000 PSI.

11. AFTER THE FINAL ADJUSTMENT OF THE DIAGONALS, THE GUYS AND
    JACKS WILL BE REMOVED AND THE GATE SHOULD RETURN TO A
    PLUMB POSITION. A DEFLECTION OF PLUS OR MINUS 1/8" WILL
    BE PERMITTED AT THE GATE'S FREE END.

12. FINAL STRESSES, UNLESS OTHERWISE APPROVED BY THE CONTRACTING
    OFFICER, SHALL BE 6,000 PSI MINIMUM AND 16,000 PSI MAXIMUM FOR
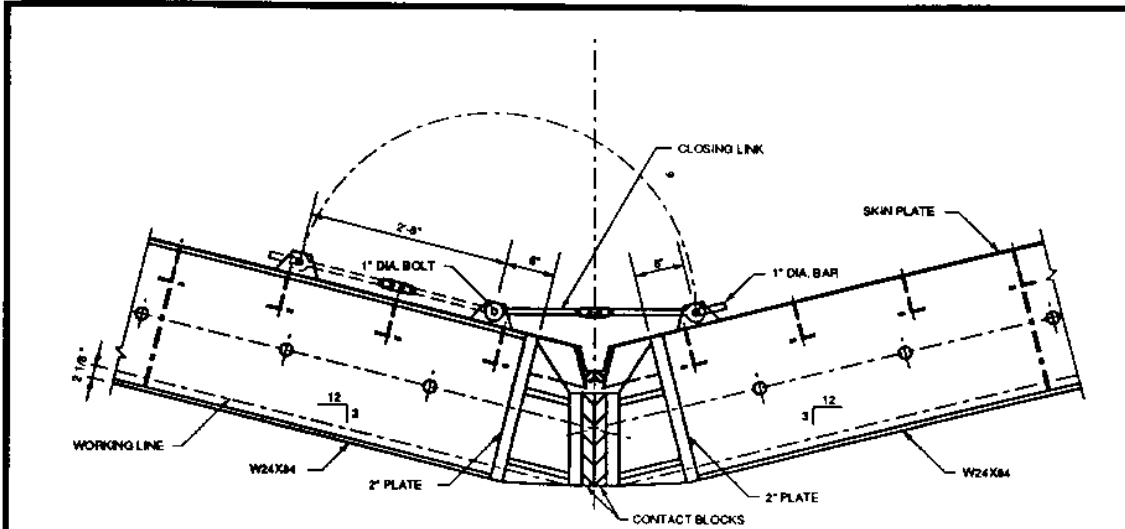   ALL DIAGONALS.

NOTE: THE DETAILS SHOWN ON THIS DRAWING WERE
USED IN A PAST CORPS OF ENGINEERS DESIGN.
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GATE CLOSURES
LOCAL FLOOD PROTECTION PROJECTS
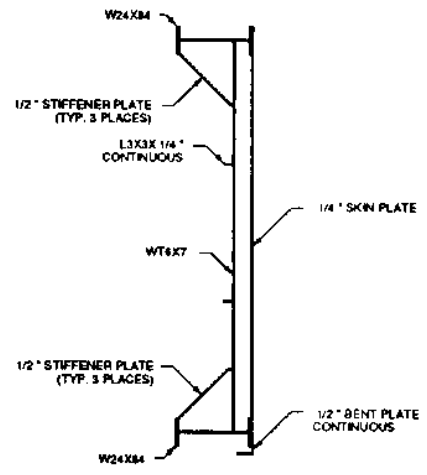TYPICAL DIAGONAL PRESTRESSING
NOTES II

DEPARTMENT OF THE ARMY
U.S. ARMY CORPS OF ENGINEERS
WASHINGTON, D.C.

PLATE 138

PLAN
SCALE: "=1'-0"

ELEVATION
SCALE: 1/2"=1'-0"

NOTE: SECTION A-A IS SHOWN ON PLATE 15

NOTES: 1. HINGE, SEAL, AND LATCH DETAILS USED FOR
MITER GATES ARE SIMILAR TO THOSE SHOWN
ON PLATES 4 THROUGH 12 FOR SWING GATES.
2. TYPICAL DIAGONAL PRESTRESSING NOTES ARE
SHOWN ON PLATES 13A & 13B.

NOTE: THE DETAILS SHOWN ON THIS DRAWING WERE
USED IN A PAST CORPS OF ENGINEERS DESIGN.
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GATE CLOSURES
LOCAL FLOOD PROTECTION PROJECTS
TYPICAL MITER GATE

DEPARTMENT OF THE ARMY
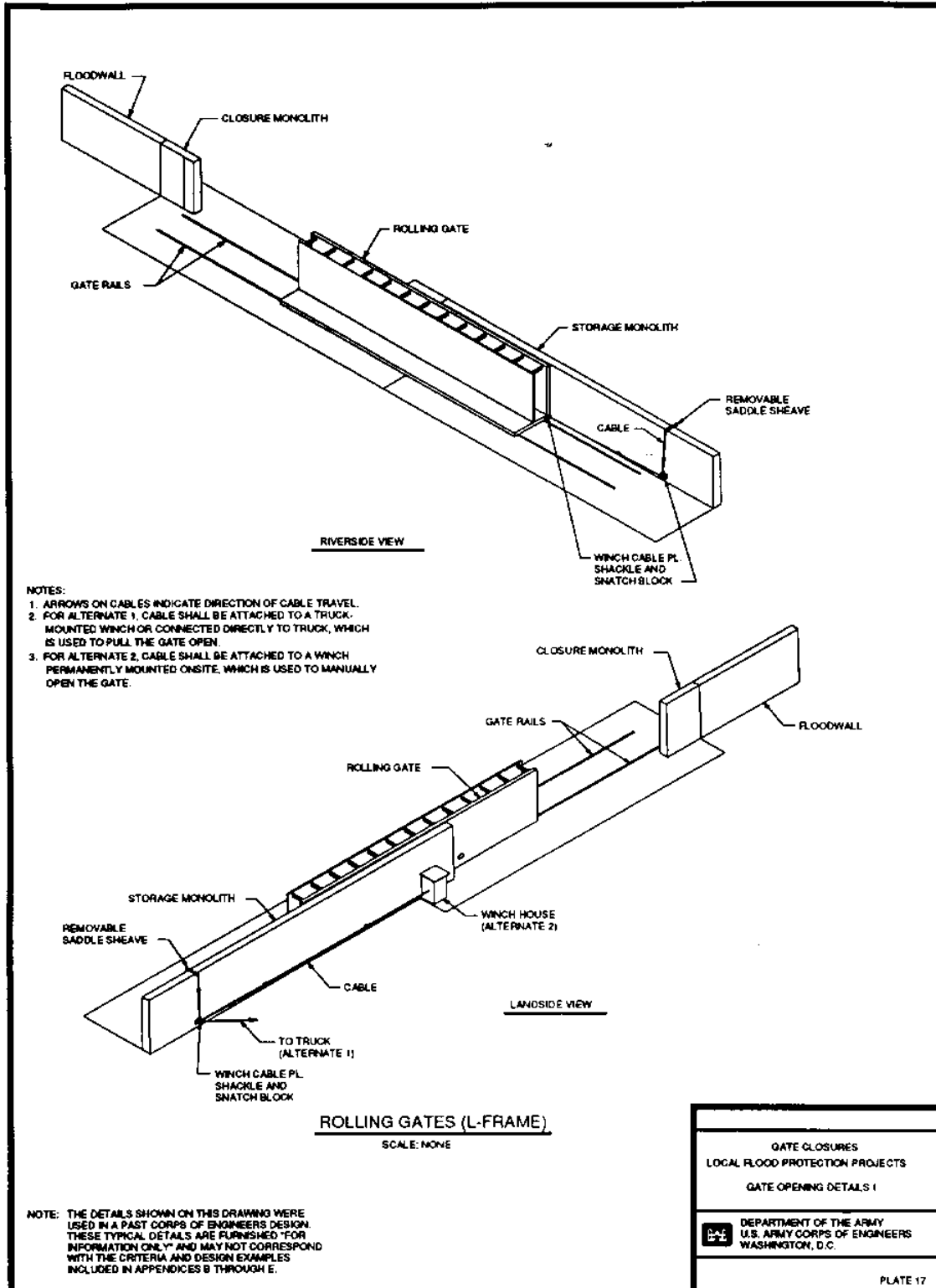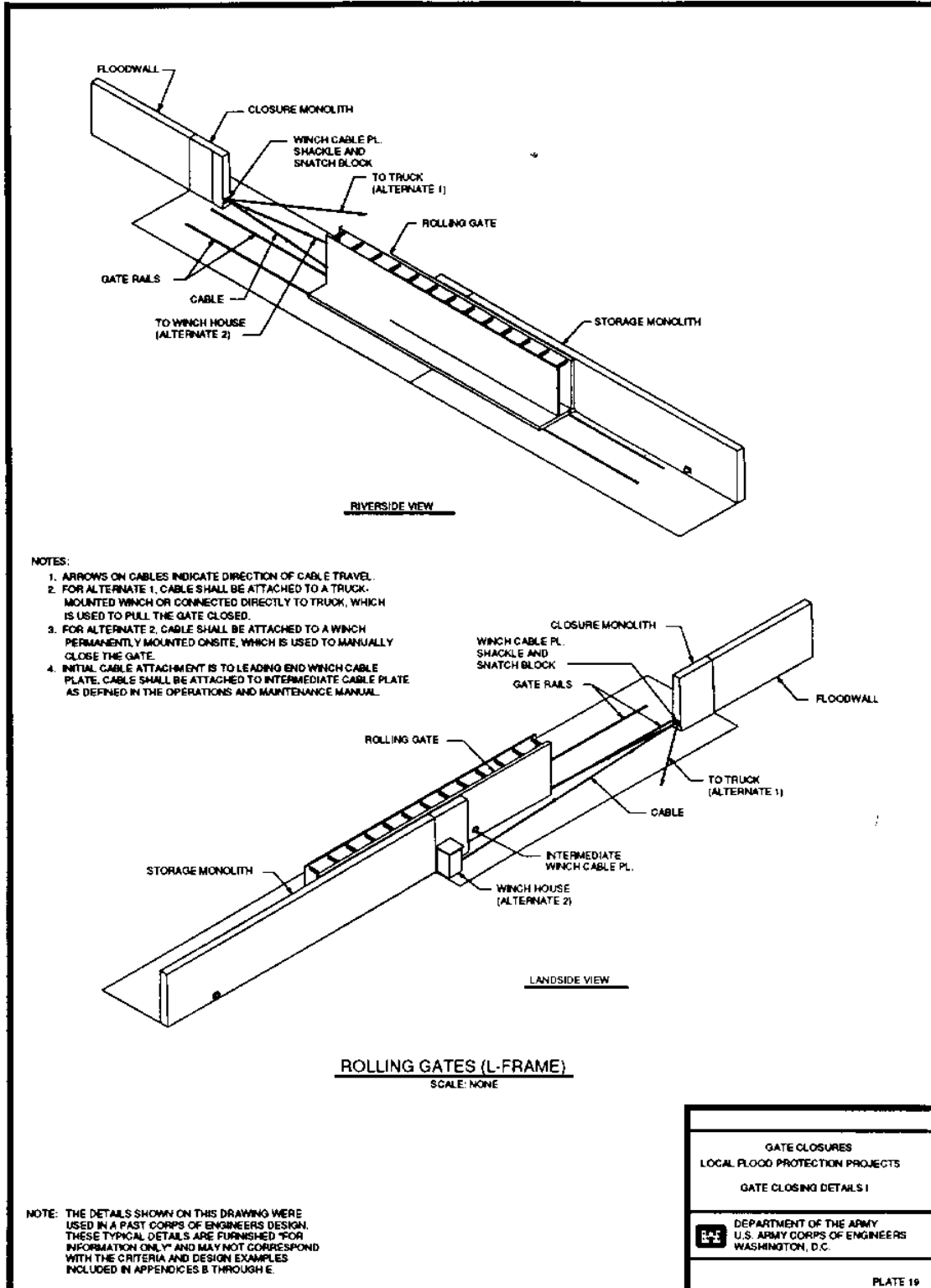U.S. ARMY CORPS OF ENGINEERS
WASHINGTON, D.C.

PLATE 14

CLOSING LINK

SKIN PLATE

1" DIA. BOLT

1" DIA. BAR

WORKING LINE

W24X84

2" PLATE

2" PLATE

W24X84

CONTACT BLOCKS

PLAN OF MITER DETAILS
SCALE: 1"=1'-0"

W24X84

1/2" STIFFENER PLATE
(TYP. 3 PLACES)

L3X3X 1/4"
CONTINUOUS

1/4" SKIN PLATE

WT6X7

1/2" STIFFENER PLATE
(TYP. 3 PLACES)

1/2" BENT PLATE
CONTINUOUS

W24X84

SECTION
A
SCALE: 1/2"=1'-0"

NOTE: THE DETAILS SHOWN ON THIS DRAWING WERE
USED IN A PAST CORPS OF ENGINEERS DESIGN.
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GATE CLOSURES
LOCAL FLOOD PROTECTION PROJECTS

TYPICAL MITER GATE DETAILS I

DEPARTMENT OF THE ARMY
U.S. ARMY CORPS OF ENGINEERS
WASHINGTON, D.C.

PLATE 15
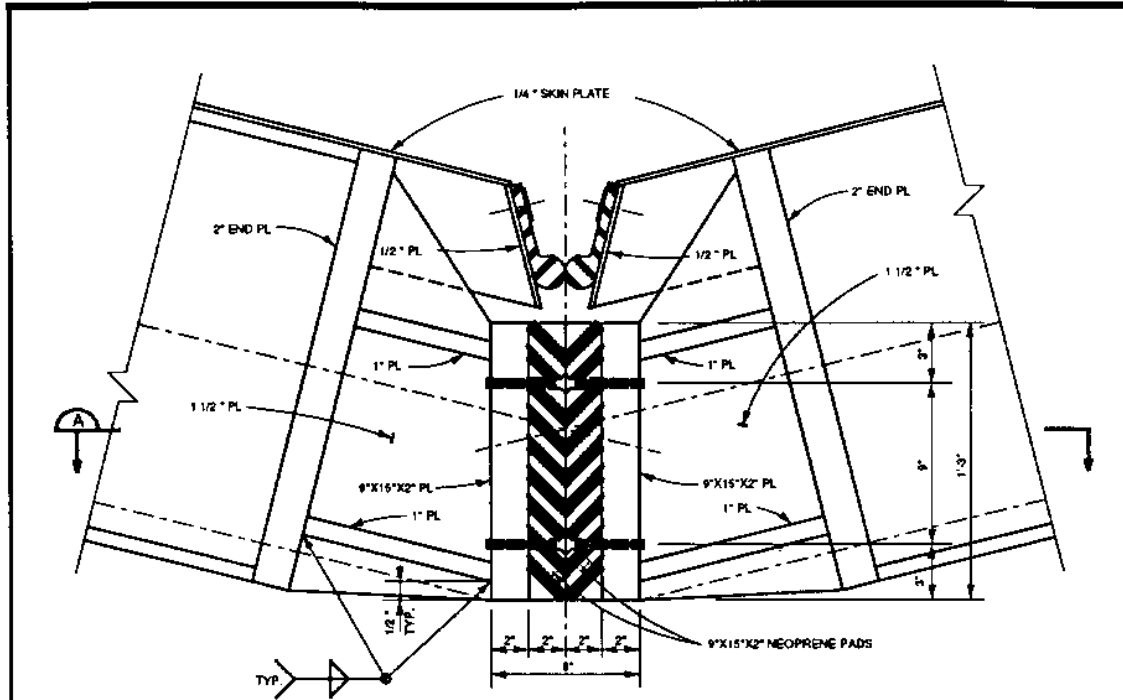
PLAN OF MITER CONTACT BLOCKS
SCALE: 3"=1'-0"
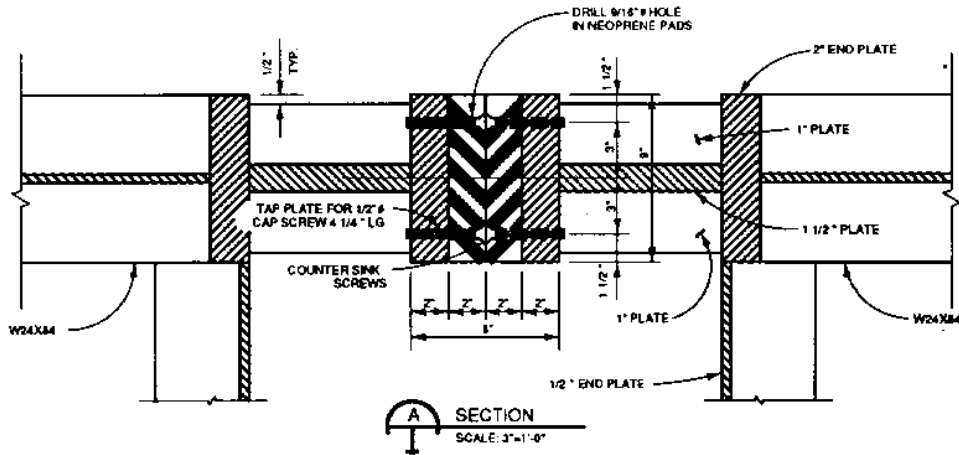
SECTION
SCALE: 3"=1'-0"

NOTES: 1. HINGE, SEAL, AND LATCH DETAILS USED FOR
MITER GATES ARE SIMILAR TO THOSE SHOWN
FOR SWING GATES.
2. TYPICAL DIAGONAL PRESTRESSING NOTES ARE
SHOWN ON PLATES 13A AND 13B.

NOTE: THE DETAILS SHOWN ON THIS DRAWING WERE
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GATE CLOSURES
LOCAL FLOOD PROTECTION PROJECTS

TYPICAL MITER GATE

DEPARTMENT OF THE ARMY
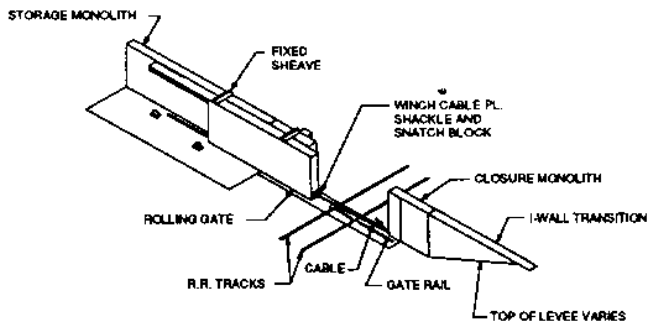U.S. ARMY CORPS OF ENGINEERS
WASHINGTON, D.C.

PLATE 18

RIVERSIDE VIEW

NOTES:
1. ARROWS ON CABLES INDICATE DIRECTION OF CABLE TRAVEL.
2. FOR ALTERNATE 1, CABLE SHALL BE ATTACHED TO A TRUCK-MOUNTED WINCH OR CONNECTED DIRECTLY TO TRUCK, WHICH IS USED TO PULL THE GATE OPEN.
3. FOR ALTERNATE 2, CABLE SHALL BE ATTACHED TO A WINCH PERMANENTLY MOUNTED ONSITE, WHICH IS USED TO MANUALLY OPEN THE GATE.

LANDSIDE VIEW

ROLLING GATES (L-FRAME)
SCALE: NONE

NOTE: THE DETAILS SHOWN ON THIS DRAWING WERE USED IN A PAST CORPS OF ENGINEERS DESIGN. THESE TYPICAL DETAILS ARE FURNISHED "FOR INFORMATION ONLY" AND MAY NOT CORRESPOND WITH THE CRITERIA AND DESIGN EXAMPLES INCLUDED IN APPENDICES B THROUGH E.
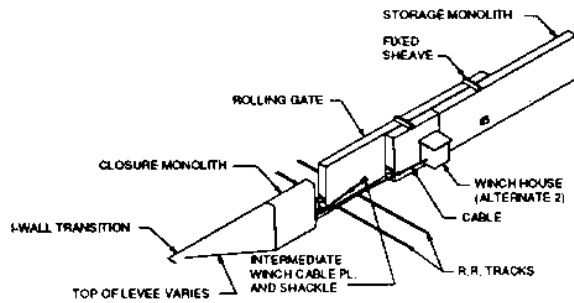
GATE CLOSURES
LOCAL FLOOD PROTECTION PROJECTS
GATE OPENING DETAILS I

DEPARTMENT OF THE ARMY
U.S. ARMY CORPS OF ENGINEERS
WASHINGTON, D.C.

PLATE 17

STORAGE MONOLITH

FIXED SHEAVE

WINCH HOUSE

CABLE

ROLLING GATE

CLOSURE MONOLITH

WINCH CABLE PL.
SHACKLE AND
SNATCH BLOCK

I-WALL TRANSITION

R.R. TRACKS

GATE RAIL

TOP OF LEVEE VARIES

RIVERSIDE VIEW

STORAGE MONOLITH

FIXED SHEAVE

ROLLING GATE

WINCH CABLE PL.
SHACKLE AND
SNATCH BLOCK

CLOSURE MONOLITH

CABLE

WINCH HOUSE
(ALTERNATE 2)
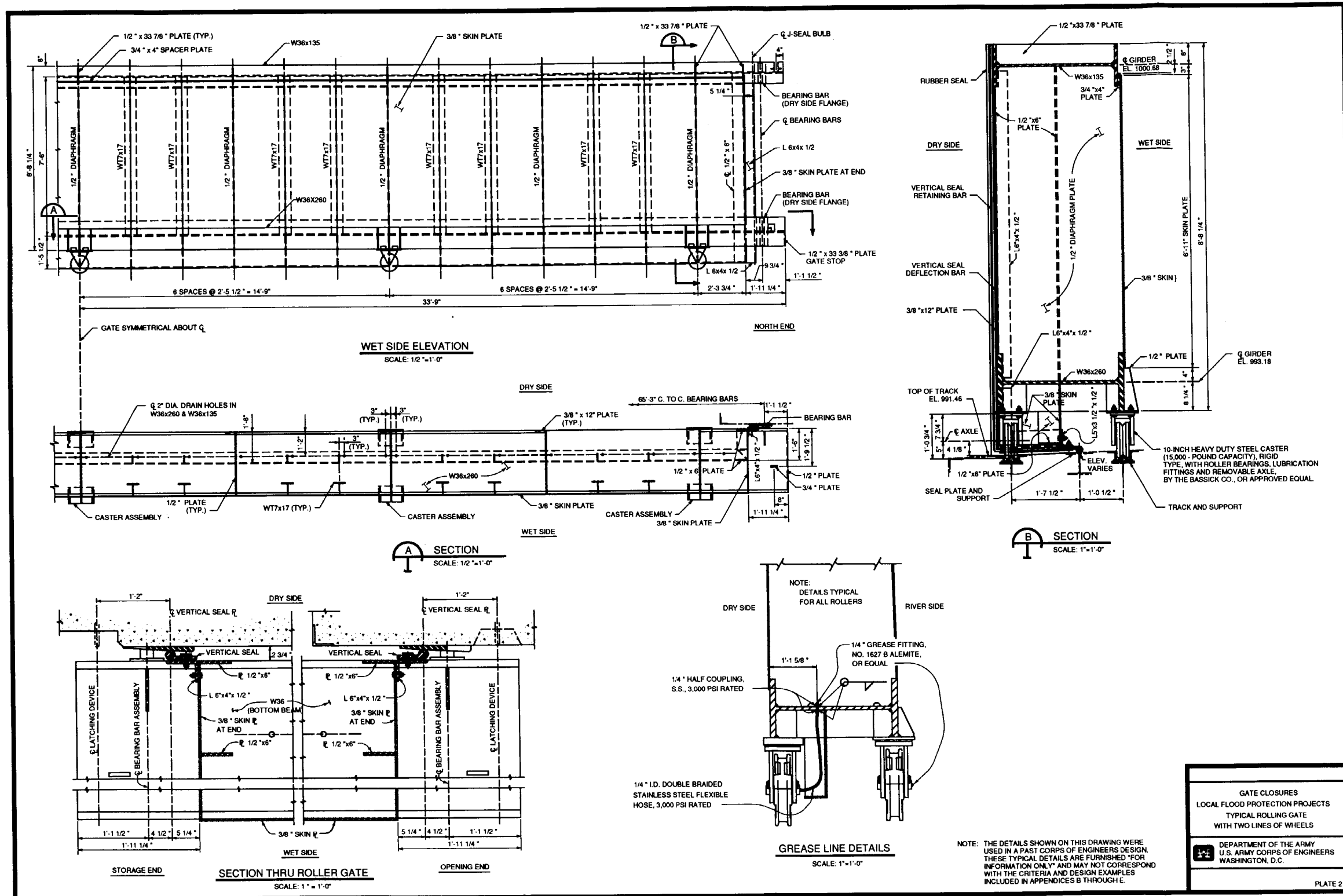
I-WALL TRANSITION

R.R. TRACKS

TOP OF LEVEE VARIES

LANDSIDE VIEW
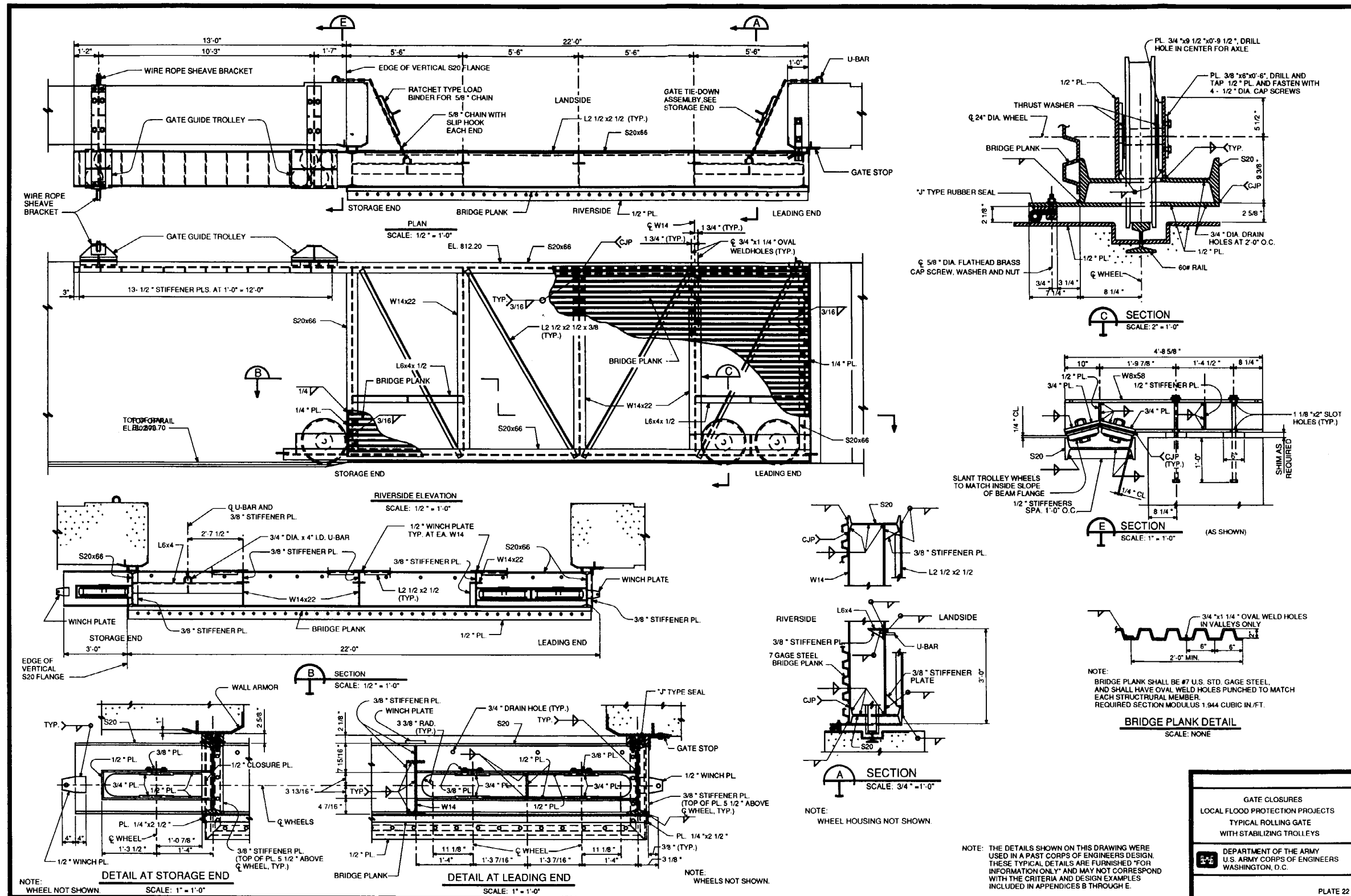
ROLLING GATE WITH STABILIZING TROLLEYS
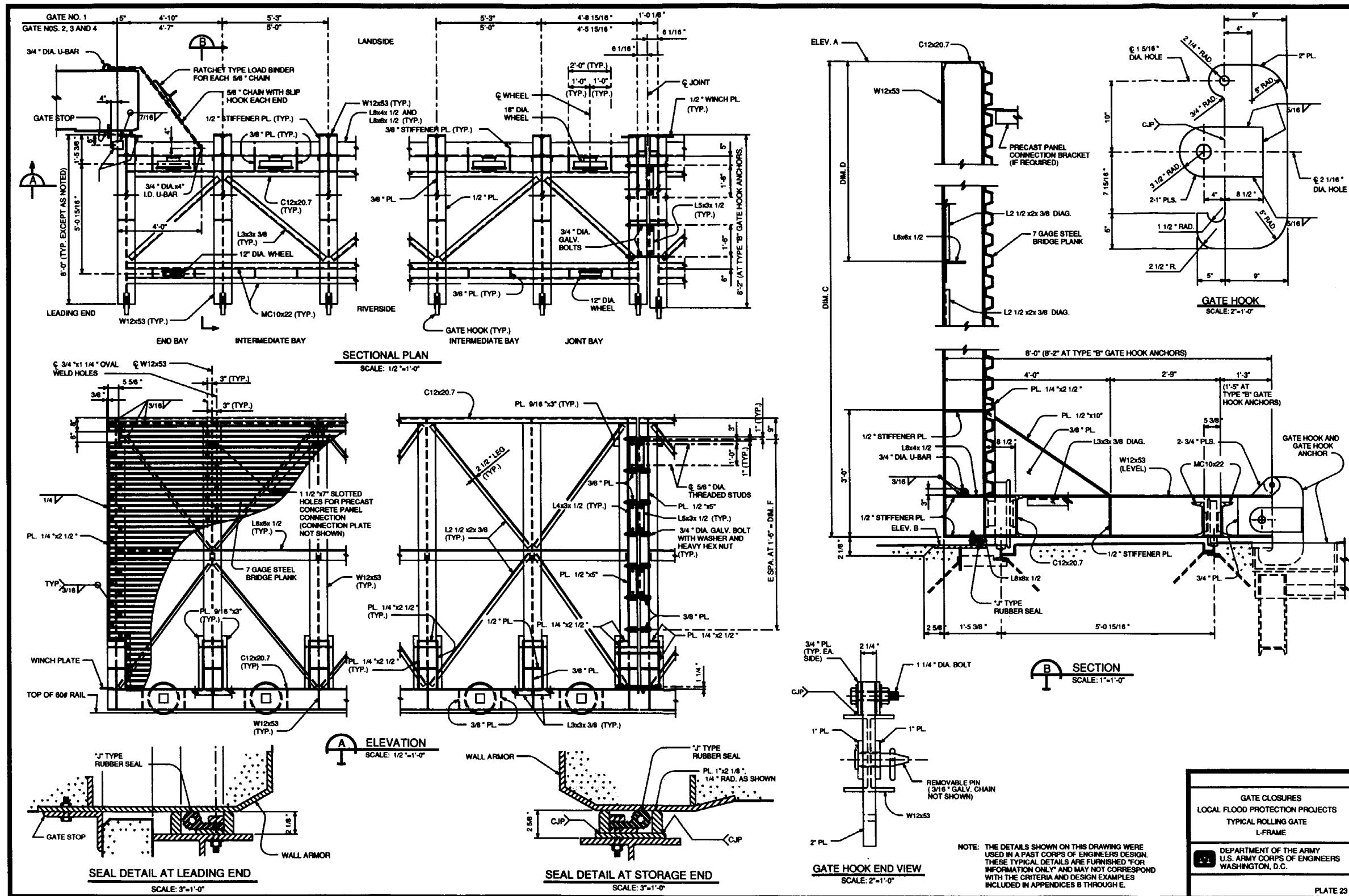SCALE: NONE

NOTES:
1. ARROWS ON CABLES INDICATE DIRECTION OF CABLE TRAVEL.
2. FOR ALTERNATE 1, CABLE SHALL BE ATTACHED TO A TRUCK-MOUNTED WINCH OR CONNECTED DIRECTLY TO TRUCK, WHICH IS USED TO PULL THE GATE OPEN.
3. FOR ALTERNATE 2, CABLE SHALL BE ATTACHED TO A WINCH PERMANENTLY MOUNTED ONSITE, WHICH IS USED TO MANUALLY OPEN THE GATE.
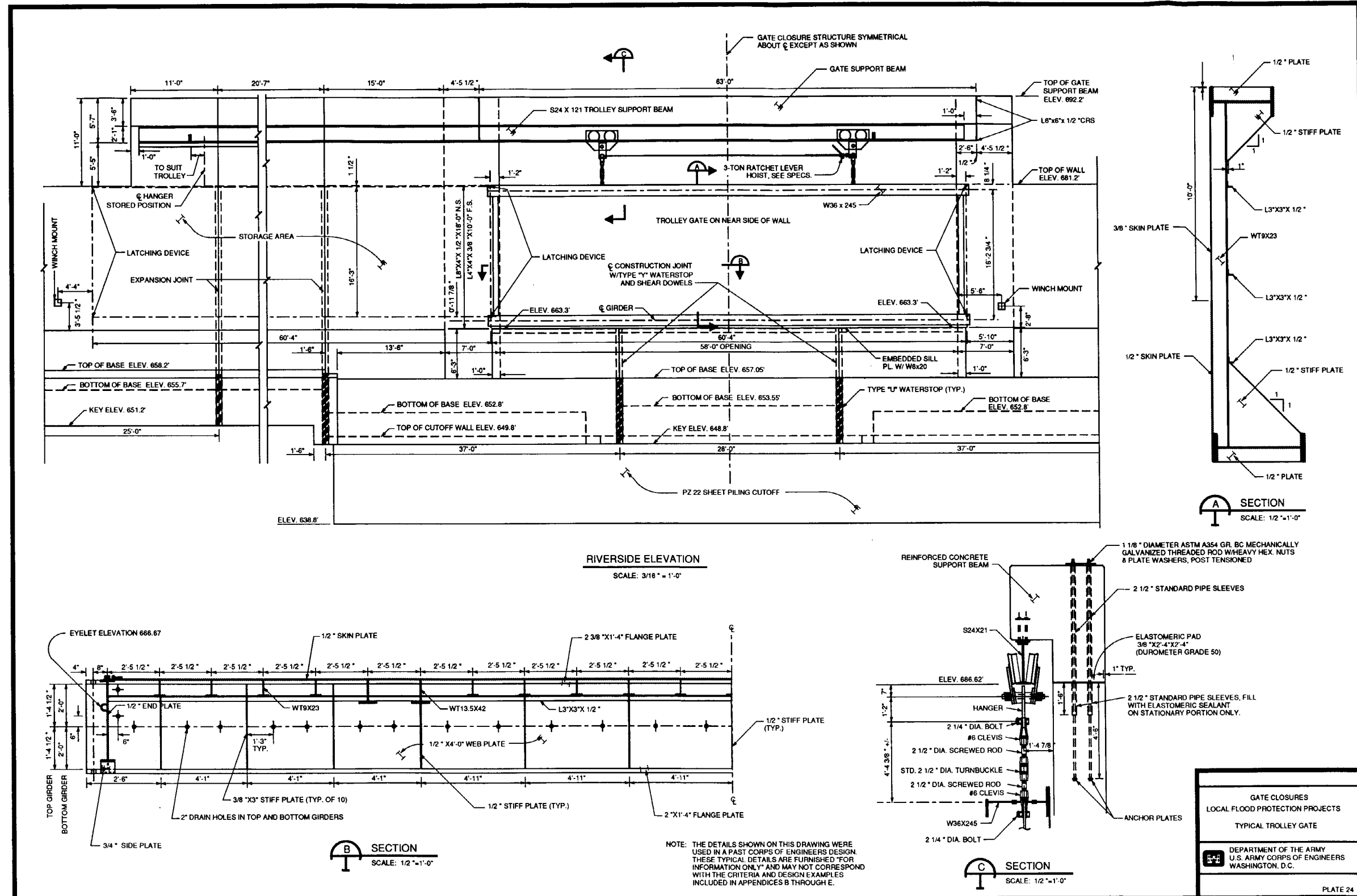
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GATE CLOSURES
LOCAL FLOOD PROTECTION PROJECTS

GATE OPENING DETAILS II

DEPARTMENT OF THE ARMY
U.S. ARMY CORPS OF ENGINEERS
WASHINGTON, D.C.

PLATE 18

RIVERSIDE VIEW

NOTES:

1. ARROWS ON CABLES INDICATE DIRECTION OF CABLE TRAVEL.
2. FOR ALTERNATE 1, CABLE SHALL BE ATTACHED TO A TRUCK-MOUNTED WINCH OR CONNECTED DIRECTLY TO TRUCK, WHICH IS USED TO PULL THE GATE CLOSED.
3. FOR ALTERNATE 2, CABLE SHALL BE ATTACHED TO A WINCH PERMANENTLY MOUNTED ONSITE, WHICH IS USED TO MANUALLY CLOSE THE GATE.
4. INITIAL CABLE ATTACHMENT IS TO LEADING END WINCH CABLE PLATE. CABLE SHALL BE ATTACHED TO INTERMEDIATE CABLE PLATE AS DEFINED IN THE OPERATIONS AND MAINTENANCE MANUAL.

LANDSIDE VIEW

ROLLING GATES (L-FRAME)
SCALE: NONE

NOTE: THE DETAILS SHOWN ON THIS DRAWING WERE USED IN A PAST CORPS OF ENGINEERS DESIGN. THESE TYPICAL DETAILS ARE FURNISHED "FOR INFORMATION ONLY" AND MAY NOT CORRESPOND WITH THE CRITERIA AND DESIGN EXAMPLES INCLUDED IN APPENDICES B THROUGH E.

GATE CLOSURES
LOCAL FLOOD PROTECTION PROJECTS

GATE CLOSING DETAILS I

DEPARTMENT OF THE ARMY
U.S. ARMY CORPS OF ENGINEERS
WASHINGTON, D.C.

PLATE 19

STORAGE MONOLITH

FIXED
SHEAVE

WINCH CABLE PL.
SHACKLE AND
SNATCH BLOCK

CLOSURE MONOLITH

I-WALL TRANSITION

ROLLING GATE

R.R. TRACKS

CABLE

GATE RAIL

TOP OF LEVEE VARIES

RIVERSIDE VIEW

STORAGE MONOLITH

FIXED
SHEAVE

ROLLING GATE

WINCH HOUSE
(ALTERNATE 2)

CLOSURE MONOLITH

CABLE

I-WALL TRANSITION

INTERMEDIATE
WINCH CABLE PL.
AND SHACKLE

R.R. TRACKS

TOP OF LEVEE VARIES

LANDSIDE VIEW

ROLLING GATE WITH STABILIZING TROLLEYS
SCALE: NONE

NOTES:
1. ARROWS ON CABLES INDICATE DIRECTION OF CABLE TRAVEL.
2. FOR ALTERNATE 1, CABLE SHALL BE ATTACHED TO A TRUCK-
   MOUNTED WINCH OR CONNECTED DIRECTLY TO TRUCK, WHICH
   IS USED TO PULL THE GATE CLOSED.
3. FOR ALTERNATE 2, CABLE SHALL BE ATTACHED TO A WINCH
   PERMANENTLY MOUNTED ONSITE, WHICH IS USED TO MANUALLY
   CLOSE THE GATE.
4. INITIAL CABLE ATTACHMENT IS TO LEADING END WINCH CABLE
   PLATE. CABLE SHALL BE ATTACHED TO INTERMEDIATE CABLE PLATE
   AS DEFINED IN THE OPERATIONS AND MAINTENANCE MANUAL.

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GATE CLOSURES
LOCAL FLOOD PROTECTION PROJECTS

GATE CLOSING DETAILS II

DEPARTMENT OF THE ARMY
U.S. ARMY CORPS OF ENGINEERS
WASHINGTON, D.C.

PLATE 20

WET SIDE ELEVATION
SCALE: 1/2"=1'-0"

1/2" x 33 7/8" PLATE (TYP.)
3/4" x 4" SPACER PLATE
W36x135
3/8" SKIN PLATE
1/2" x 33 7/8" PLATE
B
C J-SEAL BULB
BEARING BAR (DRY SIDE FLANGE)
C BEARING BARS
L 6x4x 1/2
3/8" SKIN PLATE AT END
BEARING BAR (DRY SIDE FLANGE)
1/2" x 33 3/8" PLATE GATE STOP
L 6x4x 1/2

1/2" DIAPHRAGM
WT7x17
W36X260
5 1/4"
C 1/2" x 6"

6 SPACES @ 2'-5 1/2" = 14'-9"
6 SPACES @ 2'-5 1/2" = 14'-9"
33'-9"
GATE SYMMETRICAL ABOUT C
NORTH END

A **SECTION** SCALE: 1/2"=1'-0"

DRY SIDE
WET SIDE
C 2" DIA. DRAIN HOLES IN W36x260 & W36x135
3" (TYP.) 3" (TYP.)
3/8" x 12" PLATE (TYP.)
65'-3" C. TO C. BEARING BARS
BEARING BAR
W36x260
1/2" x 6" PLATE
L 6"x4" 1/2"
1/2" PLATE
3/4" PLATE
1/2" PLATE (TYP.)
WT7x17 (TYP.)
CASTER ASSEMBLY
3/8" SKIN PLATE
CASTER ASSEMBLY
3/8" SKIN PLATE

B **SECTION** SCALE: 1"=1'-0"

RUBBER SEAL
DRY SIDE
WET SIDE
1/2" x33 7/8" PLATE
C GIRDER EL. 1000.68
W36x135
3/4" x4" PLATE
1/2" x6" PLATE
VERTICAL SEAL RETAINING BAR
L 6"x4"x 1/2"
1/2" DIAPHRAGM PLATE
6'-11" SKIN PLATE
3/8" SKIN
VERTICAL SEAL DEFLECTION BAR
3/8" x12" PLATE
L 6"x4"x 1/2"
W36x260
1/2" PLATE
C GIRDER EL 993.18
TOP OF TRACK EL. 991.46
C AXLE
3/8" SKIN PLATE
LS"x3 1/2" x 1/2"
1/2" x6" PLATE
ELEV. VARIES
SEAL PLATE AND SUPPORT
TRACK AND SUPPORT
10-INCH HEAVY DUTY STEEL CASTER (15,000-POUND CAPACITY), RIGID TYPE, WITH ROLLER BEARINGS, LUBRICATION FITTINGS AND REMOVABLE AXLE, BY THE BASSICK CO., OR APPROVED EQUAL

SECTION THRU ROLLER GATE
SCALE: 1"=1'-0"

DRY SIDE
C VERTICAL SEAL R
VERTICAL SEAL
C 1/2" x6"
L 6"x4"x 1/2"
W36 (BOTTOM BEAM)
3/8" SKIN R AT END
C LATCHING DEVICE
C BEARING BAR ASSEMBLY
1/2"x6"
3/8" SKIN R AT END
R 1/2" x6"
1/2" x6"
WET SIDE
STORAGE END
OPENING END

GREASE LINE DETAILS
SCALE: 1"=1'-0"

NOTE: DETAILS TYPICAL FOR ALL ROLLERS
DRY SIDE
RIVER SIDE
1/4" GREASE FITTING, NO. 1627 B ALEMITE, OR EQUAL
1/4" HALF COUPLING, S.S., 3,000 PSI RATED
1/4" I.D. DOUBLE BRAIDED STAINLESS STEEL FLEXIBLE HOSE, 3,000 PSI RATED
1'-1 5/8"

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GATE CLOSURES
LOCAL FLOOD PROTECTION PROJECTS
TYPICAL ROLLING GATE
WITH TWO LINES OF WHEELS

DEPARTMENT OF THE ARMY
U.S. ARMY CORPS OF ENGINEERS
WASHINGTON, D.C.

PLATE 21

PLAN
SCALE: 1/2" = 1'-0"

RIVERSIDE ELEVATION
SCALE: 1/2" = 1'-0"

SECTION B
SCALE: 1/2" = 1'-0"

DETAIL AT STORAGE END
SCALE: 1" = 1'-0"

DETAIL AT LEADING END
SCALE: 1" = 1'-0"

SECTION C
SCALE: 2" = 1'-0"

SECTION E
SCALE: 1" = 1'-0"

SECTION A
SCALE: 3/4" = 1'-0"

BRIDGE PLANK DETAIL
SCALE: NONE

NOTE:
BRIDGE PLANK SHALL BE #7 U.S. STD. GAGE STEEL,
AND SHALL HAVE OVAL WELD HOLES PUNCHED TO MATCH
EACH STRUCTURAL MEMBER.
REQUIRED SECTION MODULUS 1.944 CUBIC IN./FT.

NOTE:
WHEEL HOUSING NOT SHOWN.

NOTE:
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GATE CLOSURES
LOCAL FLOOD PROTECTION PROJECTS
TYPICAL ROLLING GATE
WITH STABILIZING TROLLEYS

DEPARTMENT OF THE ARMY
U.S. ARMY CORPS OF ENGINEERS
WASHINGTON, D.C.

PLATE 22

SECTIONAL PLAN
SCALE: 1/2"=1'-0"

GATE NO. 1
GATE NOS. 2, 3 AND 4
LANDSIDE
3/4" DIA. U-BAR
RATCHET TYPE LOAD BINDER FOR EACH 5/8" CHAIN
5/8" CHAIN WITH SLIP HOOK EACH END
GATE STOP
1/2" STIFFENER PL. (TYP.)
3/8" PL. (TYP.)
W12x53 (TYP.)
L8x4x 1/2 AND L8x8x 1/2 (TYP.)
3/8" STIFFENER PL. (TYP.)
3/4" DIA. x4" I.D. U-BAR
C12x20.7 (TYP.)
L3x3x 3/8 (TYP.)
12" DIA. WHEEL
LEADING END
W12x53 (TYP.)
MC10x22 (TYP.)
RIVERSIDE
END BAY
INTERMEDIATE BAY

C WHEEL
18" DIA. WHEEL
3/8" STIFFENER PL. (TYP.)
C JOINT
1/2" WINCH PL. (TYP.)
3/8" PL.
1/2" PL.
L5x3x 1/2 (TYP.)
3/4" DIA. GALV. BOLTS
3/8" PL. (TYP.)
12" DIA. WHEEL
GATE HOOK (TYP.)
INTERMEDIATE BAY
JOINT BAY

ELEVATION
SCALE: 1/2"=1'-0"

C 3/4"x1 1/4" OVAL WELD HOLES
C W12x53
PL. 9/16"x3" (TYP.)
C12x20.7
1 1/2"x7" SLOTTED HOLES FOR PRECAST CONCRETE PANEL CONNECTION (CONNECTION PLATE NOT SHOWN)
L8x8x 1/2 (TYP.)
PL. 1/4"x2 1/2"
7 GAGE STEEL BRIDGE PLANK
WINCH PLATE
C12x20.7 (TYP)
W12x53 (TYP.)
TOP OF 60# RAIL
L2 1/2 x2x 3/8 (TYP.)
L4x3x 1/2 (TYP.)
C 5/8" DIA. THREADED STUDS
PL. 1/2"x5"
L5x3x 1/2 (TYP.)
3/4" DIA. GALV. BOLT WITH WASHER AND HEAVY HEX NUT (TYP.)
PL. 1/2"x5"
3/8" PL.
PL. 1/4"x2 1/2"
1/2" PL.
PL. 1/4"x2 1/2"
3/8" PL.
L3x3x 3/8 (TYP.)

ELEV. A
C12x20.7
W12x53
PRECAST PANEL CONNECTION BRACKET (IF REQUIRED)
L8x8x 1/2
L2 1/2 x2x 3/8 DIAG.
7 GAGE STEEL BRIDGE PLANK
L2 1/2 x2x 3/8 DIAG.
DIM. D
DIM. C

GATE HOOK
SCALE: 2"=1'-0"
C 1 5/16" DIA. HOLE
2 1/4" RAD.
2" PL.
CJP
3/4" RAD.
C 2 1/16" DIA. HOLE
3 1/2" RAD.
2-1" PLS.
1 1/2" RAD.
2 1/2" R.

SECTION B
SCALE: 1"=1'-0"

8'-0" (8'-2" AT TYPE "B" GATE HOOK ANCHORS)
4'-0"
2'-9"
1'-3"
(1'-5" AT TYPE "B" GATE HOOK ANCHORS)
1/2" STIFFENER PL.
L8x4x 1/2
3/4" DIA. U-BAR
1/2" STIFFENER PL.
ELEV. B
PL. 1/4"x2 1/2"
PL. 1/2"x10"
3/8" PL.
L3x3x 3/8 DIAG.
W12x53 (LEVEL)
MC10x22
GATE HOOK AND GATE HOOK ANCHOR
C12x20.7
L8x8x 1/2
"J" TYPE RUBBER SEAL
1/2" STIFFENER PL.
3/4" PL.

SEAL DETAIL AT LEADING END
SCALE: 3"=1'-0"
"J" TYPE RUBBER SEAL
GATE STOP
WALL ARMOR

SEAL DETAIL AT STORAGE END
SCALE: 3"=1'-0"
WALL ARMOR
"J" TYPE RUBBER SEAL
PL. 1"x2 1/8", 1/4" RAD. AS SHOWN
CJP

GATE HOOK END VIEW
SCALE: 2"=1'-0"
3/4" PL. (TYP. EA. SIDE)
2 1/4"
1 1/4" DIA. BOLT
CJP
1" PL.
1" PL.
REMOVABLE PIN (3/16" GALV. CHAIN NOT SHOWN)
W12x53
2" PL.

NOTE: THE DETAILS SHOWN ON THIS DRAWING WERE USED IN A PAST CORPS OF ENGINEERS DESIGN. THESE TYPICAL DETAILS ARE FURNISHED "FOR INFORMATION ONLY" AND MAY NOT CORRESPOND WITH THE CRITERIA AND DESIGN EXAMPLES INCLUDED IN APPENDICES B THROUGH E.

GATE CLOSURES
LOCAL FLOOD PROTECTION PROJECTS
TYPICAL ROLLING GATE
L-FRAME

DEPARTMENT OF THE ARMY
U.S. ARMY CORPS OF ENGINEERS
WASHINGTON, D.C.

PLATE 23

RIVERSIDE ELEVATION
SCALE: 3/16" = 1'-0"

SECTION B
SCALE: 1/2"=1'-0"

SECTION A
SCALE: 1/2"=1'-0"

SECTION C
SCALE: 1/2"=1'-0"

NOTE: THE DETAILS SHOWN ON THIS DRAWING WERE
USED IN A PAST CORPS OF ENGINEERS DESIGN.
THESE TYPICAL DETAILS ARE FURNISHED "FOR
INFORMATION ONLY" AND MAY NOT CORRESPOND
WITH THE CRITERIA AND DESIGN EXAMPLES
INCLUDED IN APPENDICES B THROUGH E.

GATE CLOSURES
LOCAL FLOOD PROTECTION PROJECTS

TYPICAL TROLLEY GATE

DEPARTMENT OF THE ARMY
U.S. ARMY CORPS OF ENGINEERS
WASHINGTON, D.C.

PLATE 24

Appendix A
Checklist for Structural Design of Closure Structures

A-1. Reconnaissance Phase

a. Visit site with design team.

b. With project design engineer, determine general opening size.

c. With project design engineer, geotechnical engineer, hydraulic engineer, construction engineer, and operations engineer, assess potential alternatives using historical data and engineering judgment as to their practicality to construct, operate, and maintain.

(1) Ungated closure, sandbag closures, ramps, etc.

(2) Stoplog closure.

(3) Gated closure.

d. Identify structural engineering and design efforts and costs for the feasibility phase.

A-2. Feasibility Phase

a. Request foundation information from geotechnical engineer.

(1) Angle of internal friction.

(2) Cohesion.

(3) Adhesion.

(4) Unit weight of soil.

(5) Allowable bearing pressure.

(6) Modulus of subgrade reaction.

(7) Top of rock elevation.

(8) Groundwater elevation.

(9) Pile foundation data.

b. Determine design criteria, design assumptions, and load cases.

(1) EM 1110-2-2104.

(2) EM 1110-2-2105.

(3) EM 1110-2-2703.

(4) EM 1110-2-2502.

(5) EM 1110-2-2906.

c. With project design engineer, refine opening dimensions.

(1) Reduce width to minimum.

(2) Raise sill elevation as high as possible.

(3) Determine sill and approach slab geometry.

(4) Determine overhead clearance.

(5) Determine sight distance and geometry.

(6) Determine requirements for sidewalks and guardrails.

d. With project design engineer, assess alternatives and propose best alternative.

(1) Stoplogs.

(2) Panels.

(3) Miter gate.

(4) Overhead trolley gate.

(5) Swing gate.

(6) Roller gate.

(7) Double gate.

(8) Other gate.

e. Prepare preliminary design to the level necessary to prepare a baseline cost estimate. Present design to project engineer to be given to cost engineer.

(1) Design main structural members.

(2) Size abutments and sill and perform stability analysis.

(3) Prepare preliminary pile design if needed.

(4) Prepare quantities.

f. Identify structural engineering and design efforts and costs for preparing design memorandum (DM) and/or plans and specifications (P&S).

g. With engineers from other Engineering Division elements and Construction Division, establish design and construction schedule.

h. Provide input for feasibility report to project engineer.

(1) Technical basis for selection of gate and abutments including general configuration, material, and foundation.

(2) Criteria and assumptions.

(3) Design computations.

(4) Brief description of any additional special analyses required for DM and/or P&S.

A-3. Design Memorandum

a. Review design criteria, design assumptions, and load cases.

(1) EM 1110-2-2104.

(2) EM 1110-2-2105.

(3) EM 1110-2-2502.

(4) EM 1110-2-2703.

(5) EM 1110-2-2906.

b. Continue design.

(1) Refine sill configuration.

(2) Design secondary structural members.

(3) Refine foundation pile design.

(4) Design reinforcing steel for abutments and sill.

(5) Design other appurtenances.

(6) Coordinate design with Safety and Security offices.

(7) Verify availability of materials.

(8) Check criteria, assumptions, and computations.

c. Provide input for design memorandum to project engineer.

(1) List basic data and criteria used.

(2) Prepare computations to determine use, adequacy, shape, and stability of critical sections.

(3) Participate in value engineering studies.

A-4. Plans and Specifications

a. Complete design.

(1) Design details.

(2) Refine and verify dimensional details.

(3) Complete drawings.

(4) Provide technical input for specifications.

(5) Prepare quantities and submit to project engineer to be forwarded to cost engineer.

(6) Check computations, plans, specifications, and quantities.

(7) Revise in accordance with review comments.

b. Participate in value engineering studies.

c. Provide technical assistance to other members of design team as requested.

A-5. Construction Phase

a. Review shop drawings.

b. Make site visits as scheduled and required.

c. Provide assistance in Requests for Information, claims, and modifications when requested.

d. Provide input into operation and maintenance manual.

e. Prepare as-built drawings.

A-6. Operations Phase

Depending on Local Cooperative Agreement and project purpose:

a. Participate in periodic inspections.

b. Modify as-built drawings.

c. Review operational drawings.

d. Correct project deficiencies as required.

e. Evaluate and design replacement features as required.

Appendix B
Stoplog Design Examples

The opening for the closure in this example is 47 ft, 4 in. wide by 4 ft, 4 in. high. The closure will be designed using aluminum stoplogs (Example B.1.a.) and steel stoplogs (Example B.1.b.).

Load Cases:

The load cases given in EM 1110-2-2502 for inland floodwalls were taken into consideration. The only significant case for design was found to be case I2.

Case I2, Maximum Flood Loading. Stoplogs are in place, water on the unprotected side is at top of closure, water on protected side is at or below the top of sill. Design stresses for aluminum shall not exceed 1.11 times the allowable stresses given in Aluminum Association, Inc. (1986). Design stresses for steel shall not exceed 1.11 times the allowable stresses given in AISC (1989).

The stoplogs are designed as simple beams spanning the openings between posts and the openings between walls and posts. Intermediate posts are designed as cantilever beams fixed at the sill.

EXAMPLE B.1.a.(ALUMINUM).

A=7.48, Ix=93.57

Iy=31.96, Sx=22.35

Weight=8.7lb/ft

Mat'l:6061-T6

STOPLOG SECTION

A=14.06, Ix=231.1

Iy=43.3, r_y=1.75

S1=53.8, S2=40.5

Weight=17.1lb/ft

Mat'l: 2014-T6
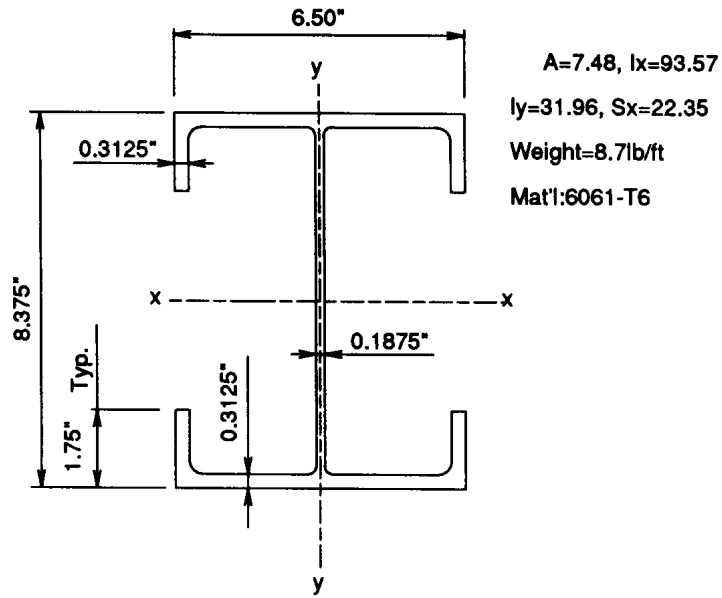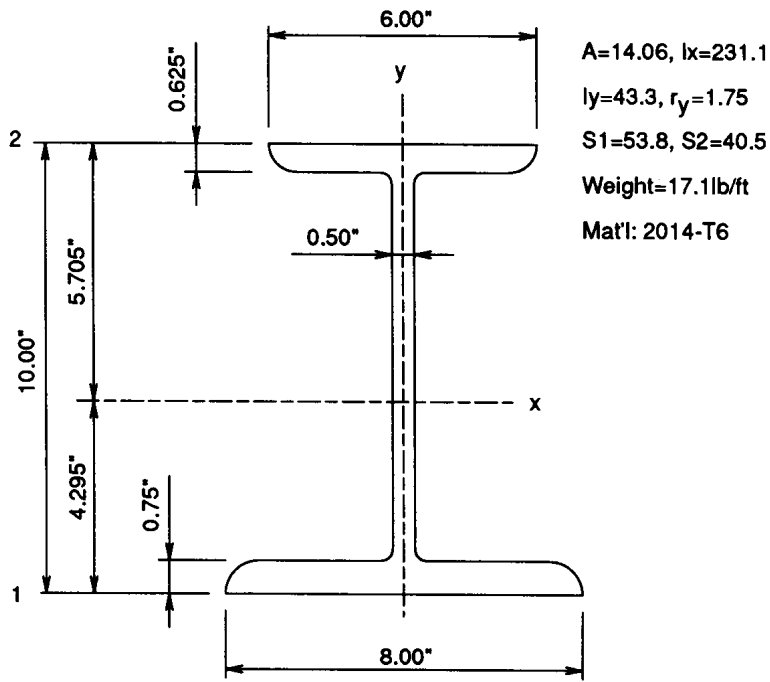
POST SECTION

EXAMPLE B.1.a.(ALUMINUM)
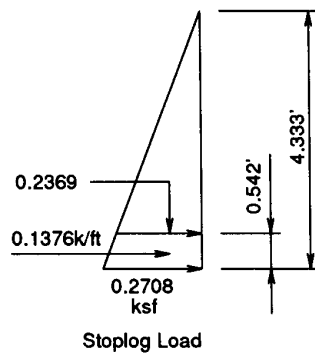
STOPLOG:

W=0.1376k/ft, L=Post Spacing=12.00'

$M=WL^2/8=2.477$k-ft=29.724k-in

V=WL/2=0.1376x12/2=0.826k

0.2369

0.1376k/ft

0.542'

4.333'

0.2708 ksf

All Specifications Listed Below Are From

Table 3.3.27 Of Aluminum Association, Inc. (1986).

Stoplog Load

Specification 1: Flexural Tension Allowable=19.00ksi

(Nonwelded Member)

Note: The Allowable Stress For Welded Aluminum

Is About 50% Less Than The Nonwelded Member

Specification 15: Compression In Flange Lips.

b/t=(1.75-0.3125)/0.3125=4.6 <6.8, F_{bc}=21ksi

Specification 16: Compression In Flange, Both Edges Supported.

b_L=1.75-0.3125=1.4375", b_f=(6.5-2x0.3125-0.1875)/2=2.844"

$b_f/3$=0.948"<b_L o.k.

b_f/t=2.844/0.3125=9.1<21, F_{bc}=21ksi

Specification 18: Compression In Web.

h/t=(8.375-2x0.3125)/0.1875=41.33<46, F_{bc}=28ksi

f_b=M/Sx=29.724/22.35=1.33ksi, F_b=1.11x19=21.09ksi>f_b o.k.

f_s=0.826/(8.375x0.18375)=0.526ksi Shear Is Negligible.

THIS STOPLOG SECTION PROVIDES AN OVERDESIGN. HOWEVER, IT IS
AVAILABLE FROM THE USING AGENCY'S INVENTORY AND CAN
BE EASILY INSTALLED BY MANUAL LABOR. IT WILL BE USED.

<u>EXAMPLE B.1.a.(ALUMINUM)</u>

10.75"

Bearing
Plate   @
Bottom

9.00"

A

Post

Wedge
@ Top

A

Pocket Armor

PLAN-POST IN POCKET

Top of Sill

Post

Wedge

Pocket Armor

Bearing Plate

C.L. of Brg.
Plate

18.00"

3.00

SECTION A-A
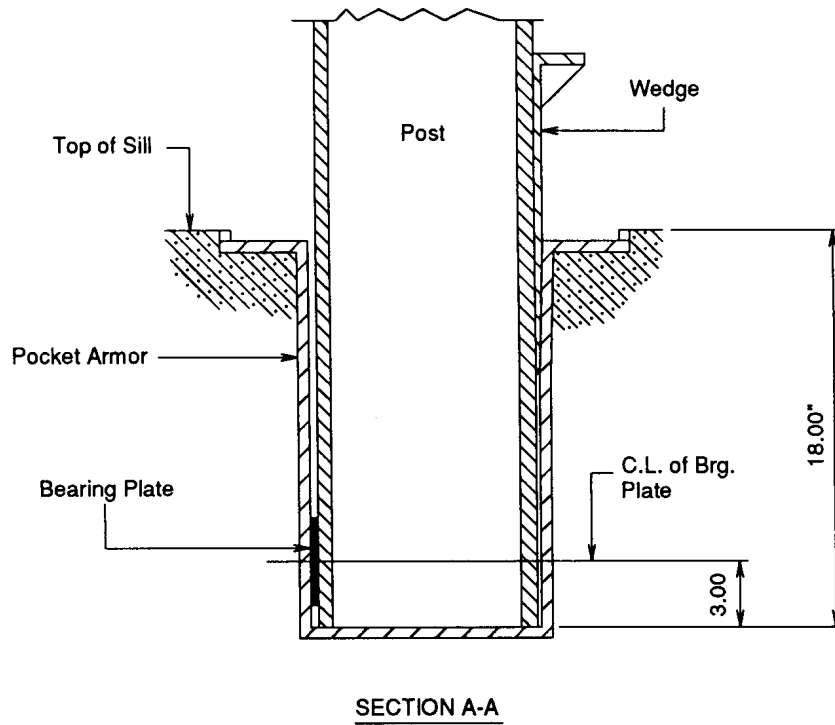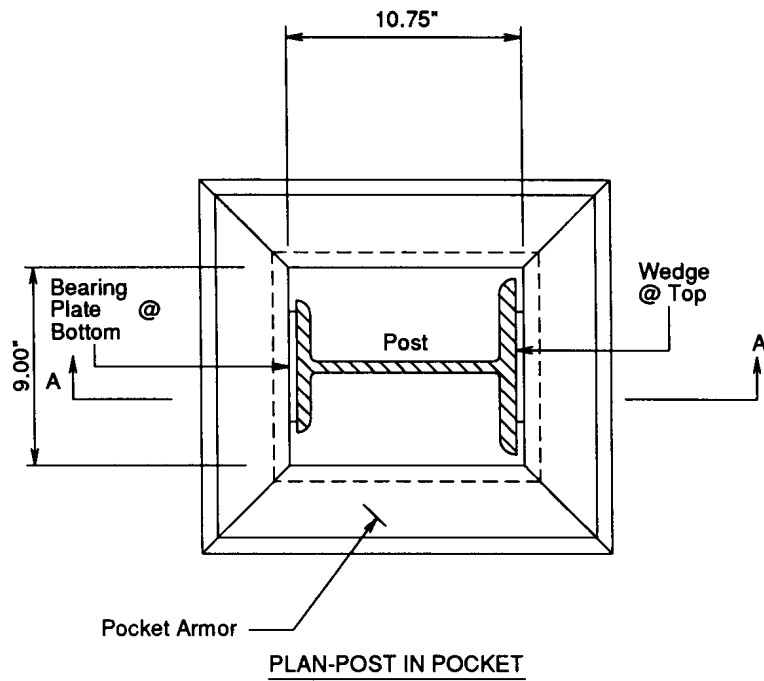
EXAMPLE B.1.a.(ALUMINUM)
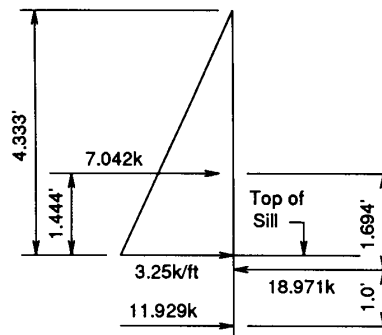
INTERMEDIATE POST:

$M=7.042 \times 1.694 = 11.93 \text{k-ft} = 143.16 \text{k-in}$

$V=11.929 \text{k}$

All Specifications Listed Below Are From

Table 3.3.8 of Aluminum Association, Inc. (1986).

POST LOADING

Specification 1: Flexural Tension Allowable=25ksi

Specification 11: Flexural Compression.

$L_b = 4.333 + 1.25 = 5.583' = 67"$, $L_b/r_y = 67/1.75 = 38 > 19$

$F_{bc} = 36.3 - 0.23 \times 38 = 27.56 \text{ksi}$

Specification 18: Compression in Web.

$h=10"$, $t_w = 0.50"$, $h/t_w = 20 < 43$, $F_{bw} = 41 \text{ksi}$

Specification 20: Web Shear.

$h/t_w = 20 < 32$, $F_s = 19 \text{ksi}$

$f_b = M/S2 = 143.16/40.5 = 3.53 \text{ksi}$, $F_b = 1.11 \times 25 = 27.75 \text{ksi} > f_b$ o.k.

$f_s = 11.929/(10 \times 0.5) = 2.386 \text{ksi}$, $F_s = 1.11 \times 19 = 21.09 \text{ksi} > f_s$ o.k.

THIS POST SECTION PROVIDES AN OVERDESIGN. HOWEVER, IT IS
AVAILABLE FROM THE USING AGENCY'S INVENTORY AND CAN BE
EASILY INSTALLED BY HAND. IT WILL BE USED.

EXAMPLE B.1.a. (ALUMINUM)

STOPLOG DEFLECTION:

$\triangle$ =5WL^4/384EI, W=0.1376/12=0.011467k/in, E=10800ksi

I=93.57 , $\triangle$ =5x0.011467(12x12)^4/(384x10800x93.57)=0.064"

$\triangle$/L=1/2250<1/200 o.k.

POST DEFLECTION:

Actual Load   Virtual Load
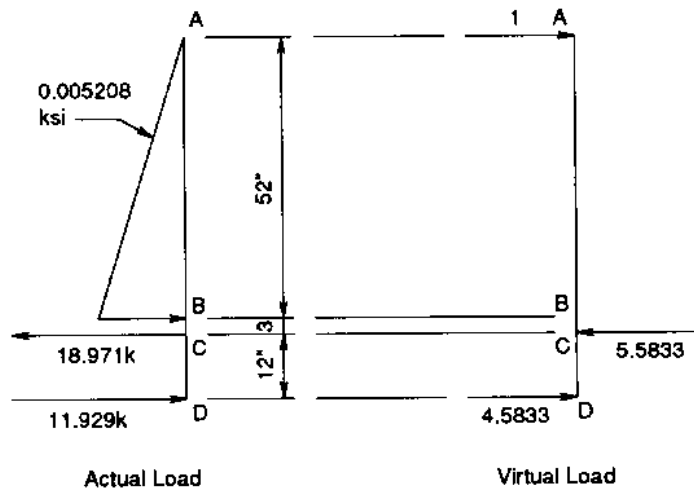
M=Moment Due To Actual Load   m=Moment Due To Virtual Load

$$\triangle A = \sum \int Mmd_y /EI$$

From A To B: M=0.000868y^3, m=y, Mm=0.000868y^4

$$\int_0^{52} Mmd_y = 66003$$

From D To C: M=11.929y, m=4.5833y, Mm=54.6742y^2

$$\int_0^{12} Mmd_y = 31493$$

From C To B: M=143.148-7.042y, m=55-y

Mm=7873.14-530.458y+7.042y^2

$$\int_0^3 Mmd_y = 21296$$

$\triangle A$ =(66003+31493+21296)/(10800x231.1)=0.048" $\triangle$/L=1/1396

o.k.

EXAMPLE B.1.b. (A36 STEEL)   All Equation Numbers In Parentheses Are

From The Specification In AISC (1989)

STOPLOG:

Trial Section S4x7.7, Ix=6.08, Sx=3.04, b_f =2.663" =0.2219'

W=Load On Bottom Log=0.5[2x0.2708-0.0625x0.2219](0.2219)

W=0.0586k/ft , M=0.0586(12)^2/8=1.055k-ft=12.66k-in

f_b =M/Sx=12.66/3.04=4.164ksi

F_b =0.66F_y =24ksi (F1-1)

$\triangle$ =5(0.0586/12)(12x12)^4 /(384x29000x6.08)=0.155"

$\triangle$ /L=1/929<1/200 o.k.

INTERMEDIATE POST:

Post Pocket Details Will Be Similar To Those

Used In Example B.1.a.

Trial Section W8x10, Ix=30.8, Sx=7.81, b_f =4" , r_T =0.99

L_b =67", L_c =76x4/6=50.67<67   (F1-2)

L_b /r_T =67/0.99=68

F_b =[2/3-36(68)^2 /(1530x10^3)]x36=20.08ksi (F1-6)

M=143.16k-in (Same As Example B.1.a.)

f_b =M/Sx=143.16/7.81=18.33ksi

1.11F_b =22.31ksi>f_b o.k.

$\triangle$ (From Example B.1.a.)=0.048"

$\triangle$ (For Steel Post)=0.048(10800/29000)(231.1/30.8)=0.134"

$\triangle$ /L=0.134/67=1/500<1/200 o.k.

USE S4x7.7 STOPLOGS AND W8x10 POSTS

B-7

Appendix C
Swing Gate Design Example

The opening for the closure in this example is 29.50 ft wide by 12.30 ft high.

Load Cases:

In accordance with EM 1110-2-2502, consideration shall be given to load cases I1 through I4. An additional case for the gate in any position, subjected to dead load only, is added for checking hinge design.

Case I1, Design Flood Loading. Gate is closed; water on the unprotected side is at the design flood elevation; water is at or below sill surface on protected side. Design stresses shall not be greater than 5/6 of stresses allowed in AISC (1989).

Case I2, Maximum Flood Loading. Same as case I1 except that water level is to top of gate on unprotected side. Design stresses shall not be greater than 1.11 times the stresses allowed in AISC (1989).

Case I3, Earthquake Loading. Water is at usual level (non-flood condition) on unprotected side; earthquake-induced forces are acting. (Note: This case is applicable to support structures only).

Case I4, Short-Duration Loading. Gate is either open, closed, or in between and is subjected to construction and/or wind loads. Design stresses shall not be greater than 1.11 times the stresses allowed in AISC (1989).

Case I5, Gate Swinging. Gate in any position, subjected to dead load only. Design stresses shall not be greater than 5/6 of stresses allowed in AISC (1989).

In this example, cases I1 and I3 are not significant and skin plate, intercostals, and girders are designed for case I2 with design stresses not greater than 1.11 times stresses allowed in AISC (1989). Case I4 is applicable for design of latching devices. Case I5 is applicable to hinge design.

The skin plate is designed as a fixed end beam, spanning between intercostals. In order to ensure that the flat plate theory is applicable, deflection will be limited to 0.4 of thickness.

The intercostals are designed as simple beams spanning between girders.

Girders are designed as simple beams, spanning between hinges on one side and wall bearings on the other side of the opening.

HORIZONTAL SECTION THRU GATE

Water Surface
Max. Flood

Top of Gate

Top Girder

0.42'

Water
Surface
Design
Flood

3.00'

Gird. Flg. Brace
Plate @ 5'-6"

5.515'

Skin Plate

Wet Side

Intercostal
Flg. Brace

11.03'

12.30'

Dry Side

5.515'

Intercostals

Gird. Flg. Brace
Plate @ 5'-6"

Continuous
"J" Seal

Bottom Girder

0.85'

Sill
Sealing
Surface
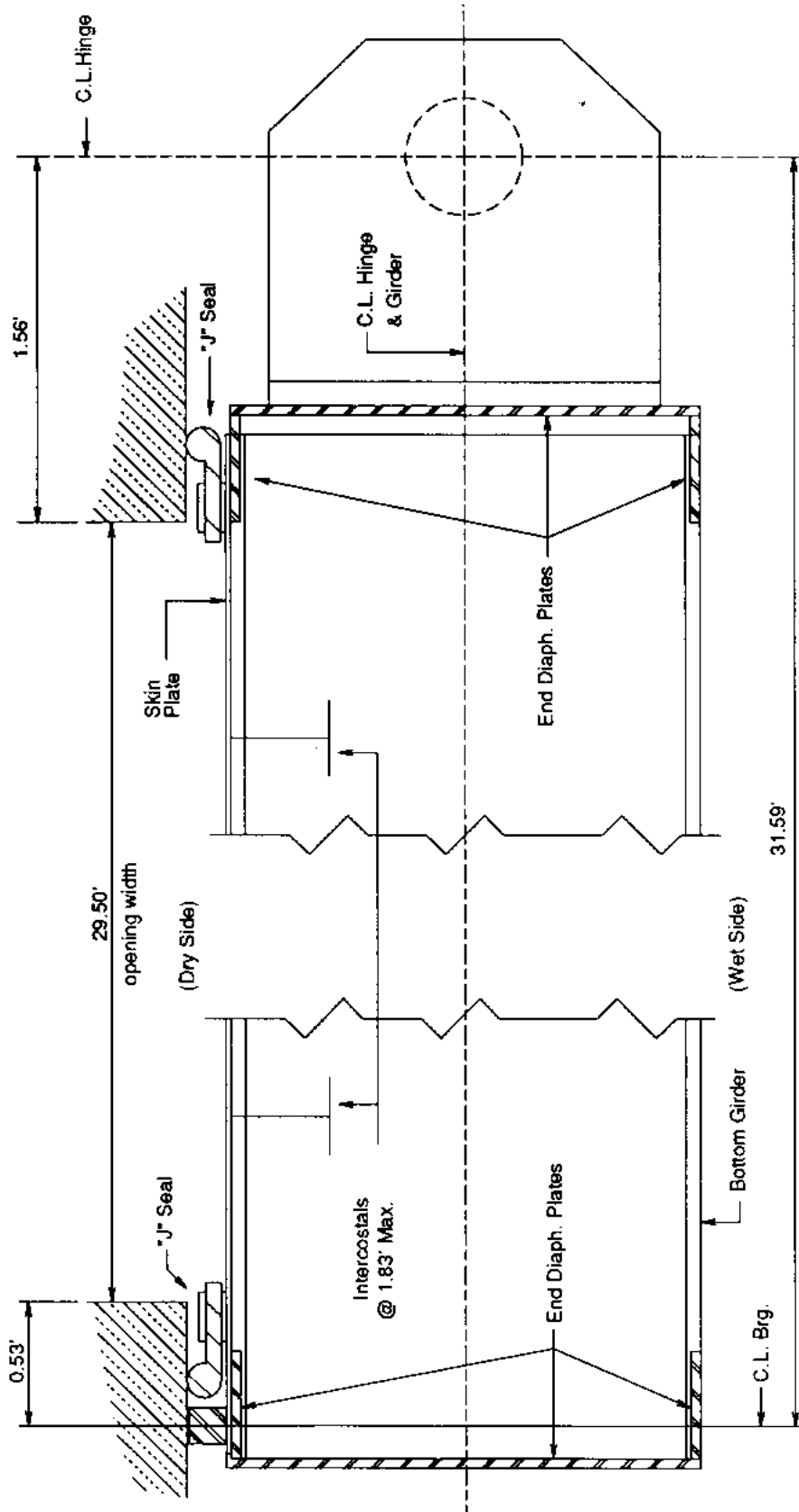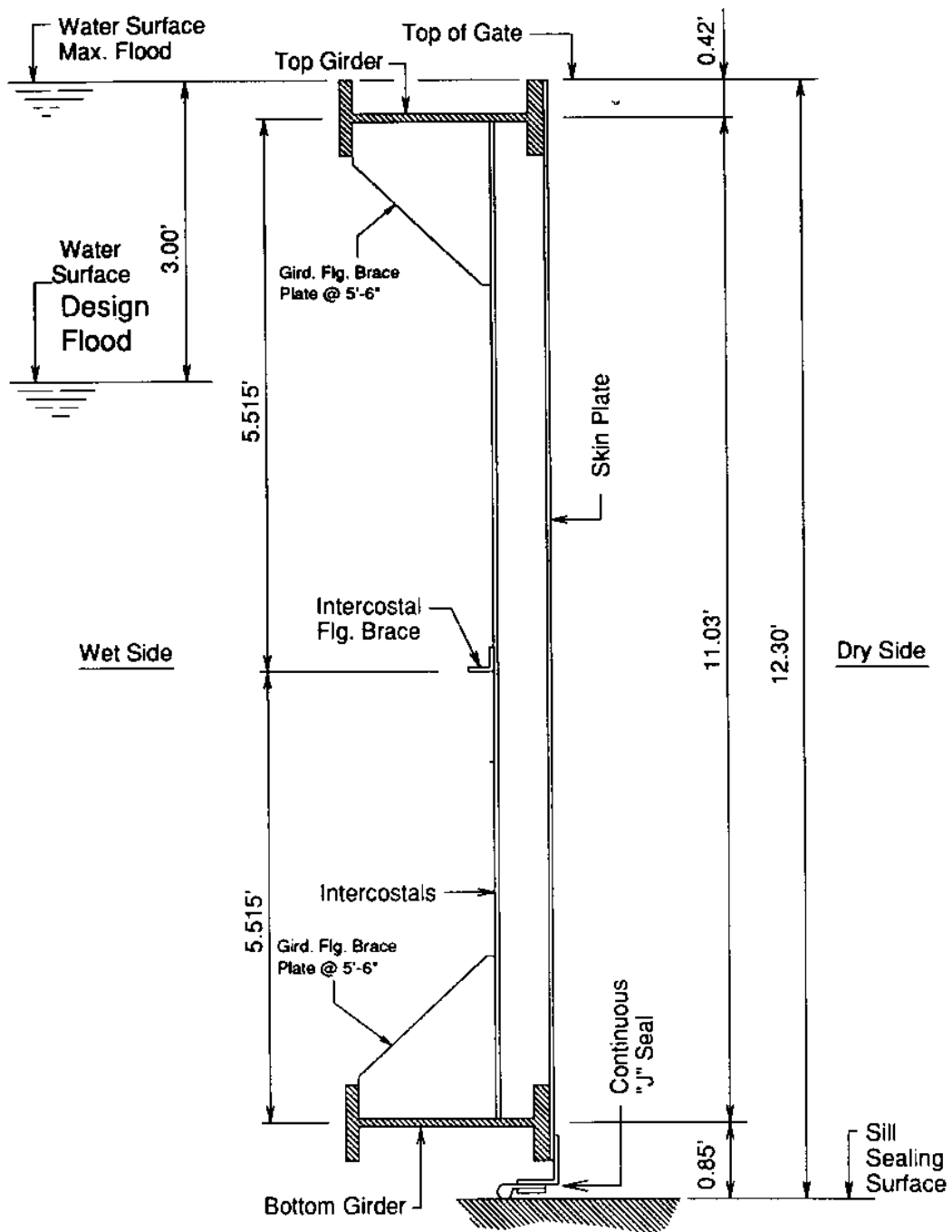
VERTICAL SECTION THRU GATE

SKIN PLATE DESIGN; LOAD CASE I2   (Material: A36 Steel)

Assuming a 9" flange for the bottom girder, the hydrostatic

pressure 6" above the flange is:

$$p=0.0625(12.3-0.85-0.375-0.5)=0.6609ksf=0.00459ksi$$

$$M=pb^2/12=0.00459(22)^2/12=0.1851k\text{-in}$$

$$F_b=1.11\times0.75F_y=1.11\times0.75\times36=30ksi$$

$$t_{min\text{-}stress}=(pb^2/2F_b)^{1/2}$$

$$t_{min\text{-}stress}=[0.00459(22)^2/(2\times30)]^{1/2}=0.1924"$$

$$defl.=pb^4/384EI; \ E=29000, \ I=t^3/12, \ defl.=0.4t$$

$$0.4t=12pb^4/384Et^3, \ t^4=pb^4/12.8E$$

$$t_{min\text{-}defl.}=[0.00459(22)^4/(12.8\times29000)]^{1/4}=0.2320"$$

USE 1/4" SKIN PLATE

For Tall Gates More Than One

Thickness May Be Required.

INTERCOSTAL DESIGN: LOAD CASE I2

Material: A36 Steel. Equations & Tables In Parentheses Are From

The Specification In AISC (1989).

Load, shear, and moment diagrams for intercostals are shown on

page C-5. Trial Section is shown on page C-6.

$$M=5.839\times22/12=10.705k\text{-ft}=128.46k\text{-in}$$

$$b_f/2t_f=4.00/(2\times0.27)=7.41$$

$$65/(F_y)^{1/2}=65/6=10.83>7.41 \ \text{Compact Section (Table B5.1)}$$

$$L_b=5.515'=66.18", \ L_b/r_T=66.18/1.03=64.25$$

$$L_c=76b_f/(F_y)^{1/2}=76\times4/6=50.67<66.18 \ \text{(F1-2)}$$

$$F_b=[2/3-F_y(L_b/r_T)^2/(1530\times10^3)]F_y \ \text{(F1-6)}$$

$$F_b=[2/3 -36(64.25)^2/(1530\times10^3)]36=20.50ksi$$

$$1.11F_b=22.78ksi$$

$$f_b=M/S=128.46/6.84=18.78ksi<22.78ksi \ \text{o.k.}$$

USE WT 5x7.5 FOR INTERCOSTALS. MAX. SPACING 1'-10"

Water Surface & Top of Gate

0.0444 ksf

0.0625 ksf/ft

0.29'

0.71'

10.365'

1.377k/ft

Intercostals

M=5.839 k-ft/ft

0.6922 ksf

-2.440k/ft

1.225'

Seal Surface

0.375'

Shear

Moment

Hydrostatic Load

LOAD, SHEAR, & MOMENT DIAGRAMS FOR INTERCOSTALS

22.00"

1/4" Skin Plate

1.200"

x ————————————————————————— x

5.245"

0.27"

4.045"

Compression
Flange

WT 5x7.5

S(min)=6.84

r_T=1.03

4.00"
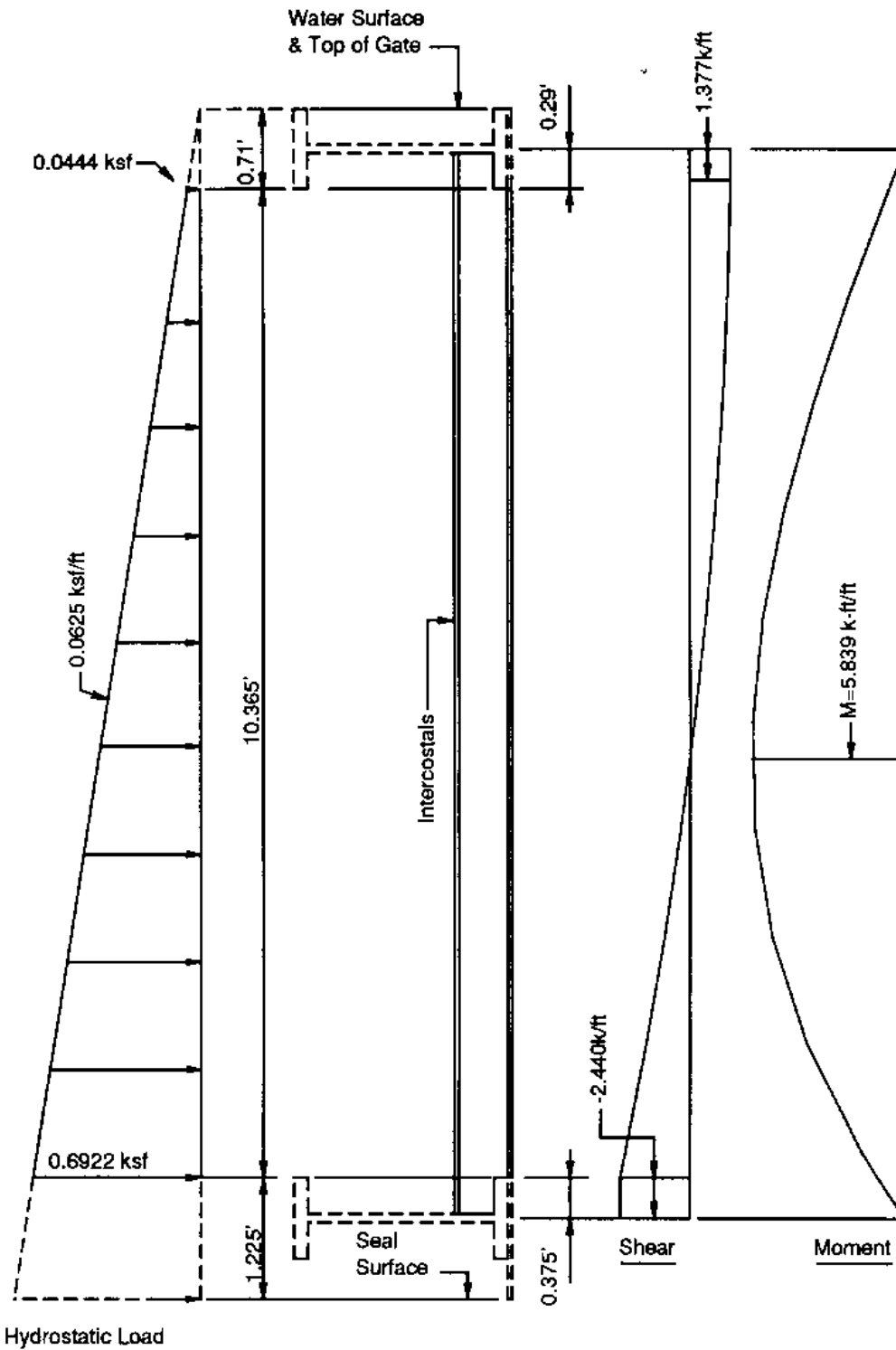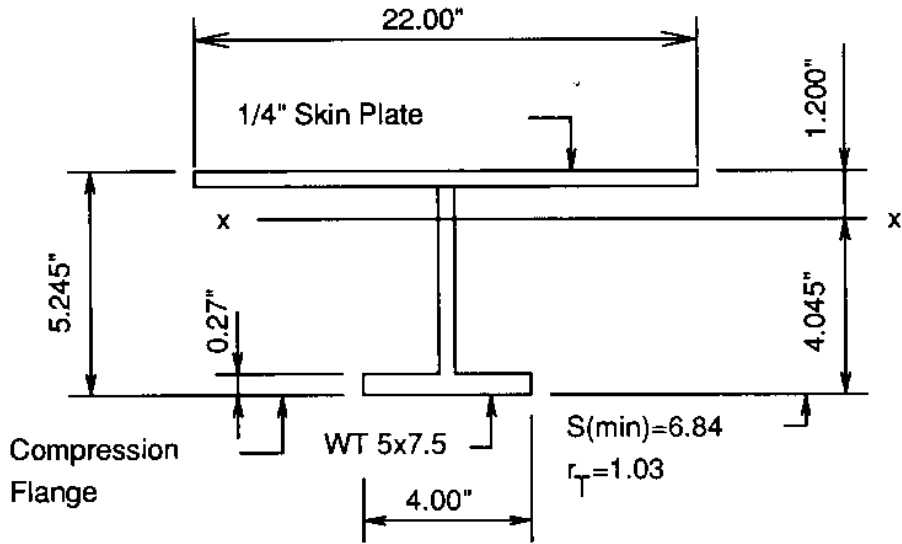
INTERCOSTAL SECTION

Girder Design   (Mat'l. A36 Steel, Equation and Table Numbers In Parentheses Are From Specification In AISC (1989).)

GIRDER LOADS

(labels on figure: 0.42', 7.78', 3.25', 4.1', 0.85, 1.393k/ft Top Gird., 4.728k/ft, 3.335k/ft Bot. Gird., 0.7688ksf)
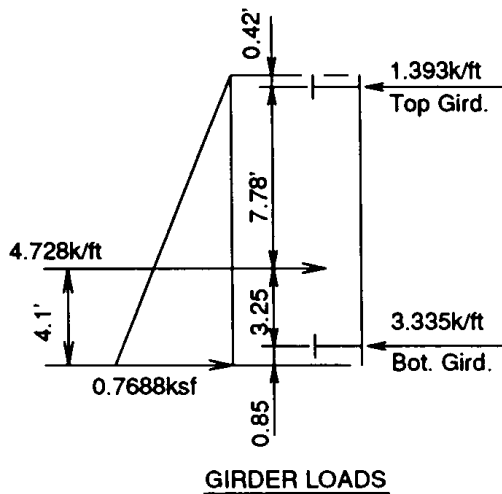
Bottom Girder

$$M=3.335(31.59)^2(1/8)$$

$$M=416.04\text{k-ft}=4992\text{k-in}$$

Trial Section W 24x84

$$Ix=2370,\ Sx=196$$

$$d/Af=3.47,\ b_f=9",\ Fy=36\text{ksi}$$

$$b_f/2t_f=5.9,\ d/t_w=51.3$$

(Table B5.1)

$$65/(Fy)^{1/2}=10.83>5.9$$

$$640/(Fy)^{1/2}=106.67>51.3$$

Flange & Web are compact

$$Lb=66",\ Lc=76b_f/(Fy)^{1/2}=114">66"\quad(F1\text{-}2)$$

$$Fb=1.11\times0.66\times Fy=26.67\text{ksi},\ f_b=M/Sx=4992/196=25.47\text{ksi}<Fb$$

USE W 24x84

See page C-18 for design with the effect of axial load & bending due to the diagonal

Top Girder

$$M=1.393(31.59)^2(1/8)=173.76\text{k-ft}=2085\text{k-in}$$

Trial Section W 24x55
$$Ix=1350,\ Sx=114,\ d/Af=6.66$$

$$b_f/2t_f=6.9,\ d/t_w=59.7,\ Lb=66"$$

Flange & Web are compact, see Table B5.1.

$$Lc=20000/(6.66\times36)=83.42">66"\quad(F1\text{-}2)$$

$$F_b=1.11(2/3)(36)=26.67\text{ksi}$$

$$f_b=2085/114=18.29\text{ksi}<26.67\text{ksi}$$

USE: W24x55
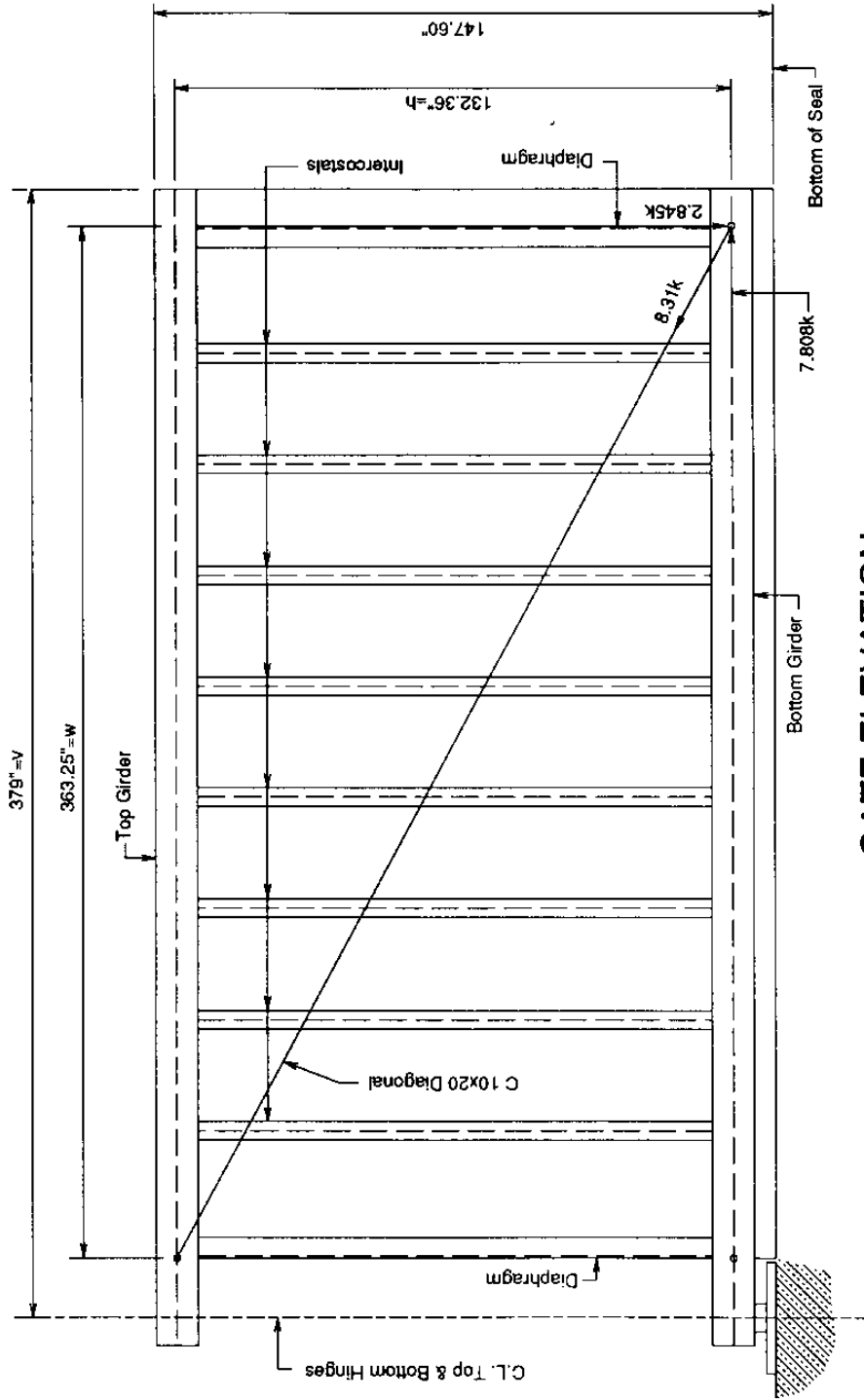
GATE ELEVATION
SHOWING DIAGONAL LOAD DUE TO D.L. TORSION

(See Page B-9 For Torsional Deflection & Diagonal Stress Calculations)

STRESS & DEFLECTION DUE TO DEAD LOAD TORSION

Methods For Determining The Shear Center And The Torsional Stiffness
Of The Gate Are Presented In EM 1110-2-2703

End View
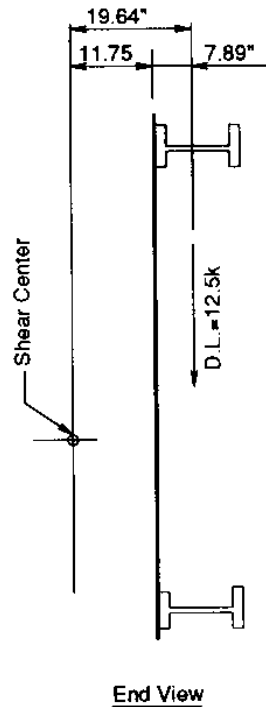
T=Torsion=12.5x19.64=245.5k-in

Z=Distance From Hinges To C.G. Of D.L.

Z=16.198'=194.38"

TZ=245.5x194.38=47720

A=Area C 10x20 Diagonal=5.88

A'=1/8 Of Sum Of Girder & Diaph. Areas

A'=10.11

$L=[(132.36)^2+ (363.25)^2]^{1/2}=386.61"$

Qo=Torsional Stiffness Of Gate =2442

t=Dist. From C.L. Of Skin To C.G. Of Diagonal

t=22.97"

Ro=2wt/vL=2x363.25x22.97/(379x386.61)

Ro=0.1139

$R=A'Ro/(A+A')$=10.11x0.1139/(5.88+10.11)

R=0.072

Q=RRoEAhv/L=Torsional Stiff. Diagonal

Q=181312, $Q+Qo$=183754

$\triangle$ =TZ/(Q+Qo)=47720/183754=0.26"=Torsional Defl. Of Gate.

Tensile Stress In Diagonal=RE$\triangle$/L=1.41ksi

Tensile Load In Diagonal=1.41x5.88=8.31k
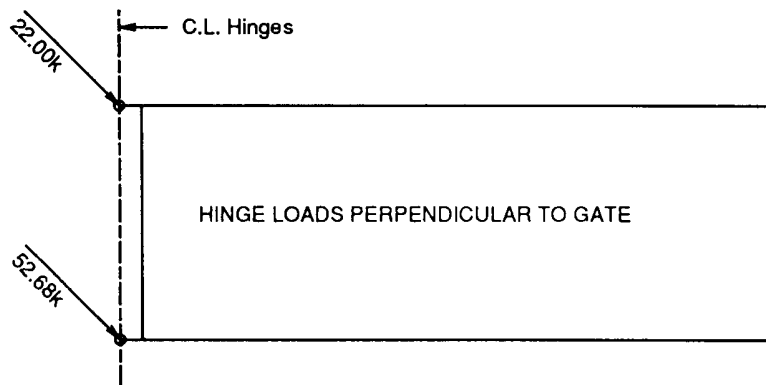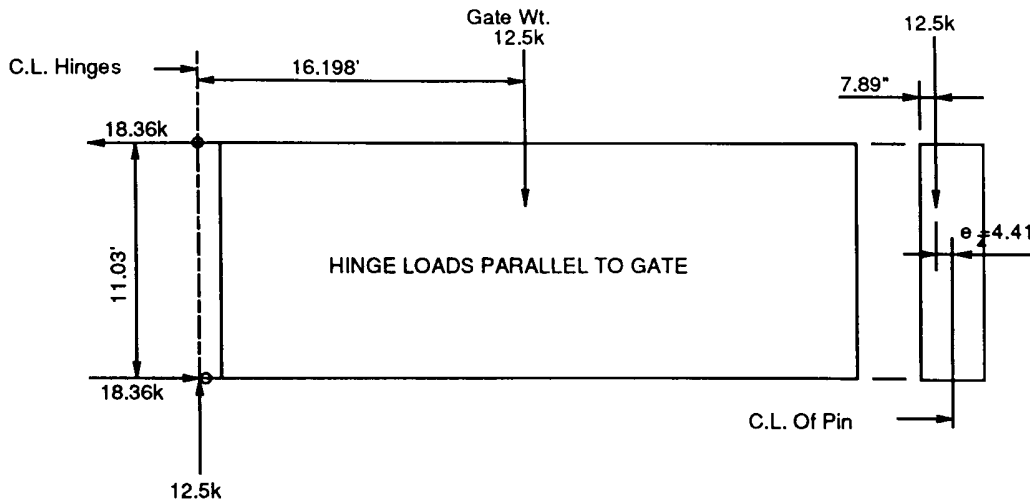
BOTTOM HINGE   PIN

P_y =52.68k, P_x =18.36k, P_z =12.5k, e_z =4.41"

Assuming a 1.75" base plate, Z = 7.85"

$M_y = P_y Z + P_z e_z$ = 468.66k-in

$M_x = P_x Z$ = 144.13 k-in

$M_r = \sqrt{M_x^2 + M_y^2}$ = 490.32 k-in

d= Pin Diameter = 5", Mat'l. A276, Fy=55 ksi

A =19.63, I = 30.68, S = 12.27, r =1.25, kL/r=2x7.85/1.25=13

1.11Fb=1.11x0.75Fy=45.83ksi, 1.11Fa = 35.43ksi (Using AISC (1989))

f_a =12.5/19.63=0.637ksi, f_b = 490.32/12.27 = 39.96 ksi

$f_a/1.11F_a + f_b/1.11F_b$= 0.890<1.000

> USE 5" DIAMETER PIN

BASE PLATE & BASE PLATE BOLTS:

16"x1.75" Sq.Plate

2.75 5.25" 5.25" 2.75

2.75

5.25"

16.00"

5.25"

2.75

1.5" Dia. Bolts

2.22"

2.22"

Face Equiv. Sq. Pin

5.00" Dia. Pin

BASE PLATE PLAN

13.25-x

x 8-x

16.00"

Sc = 263.10

N.A.

As = 3.534

As = 5.301

St = 77.00

5.00"

3.00 10.25"

2.245ksi

Brg. Press.
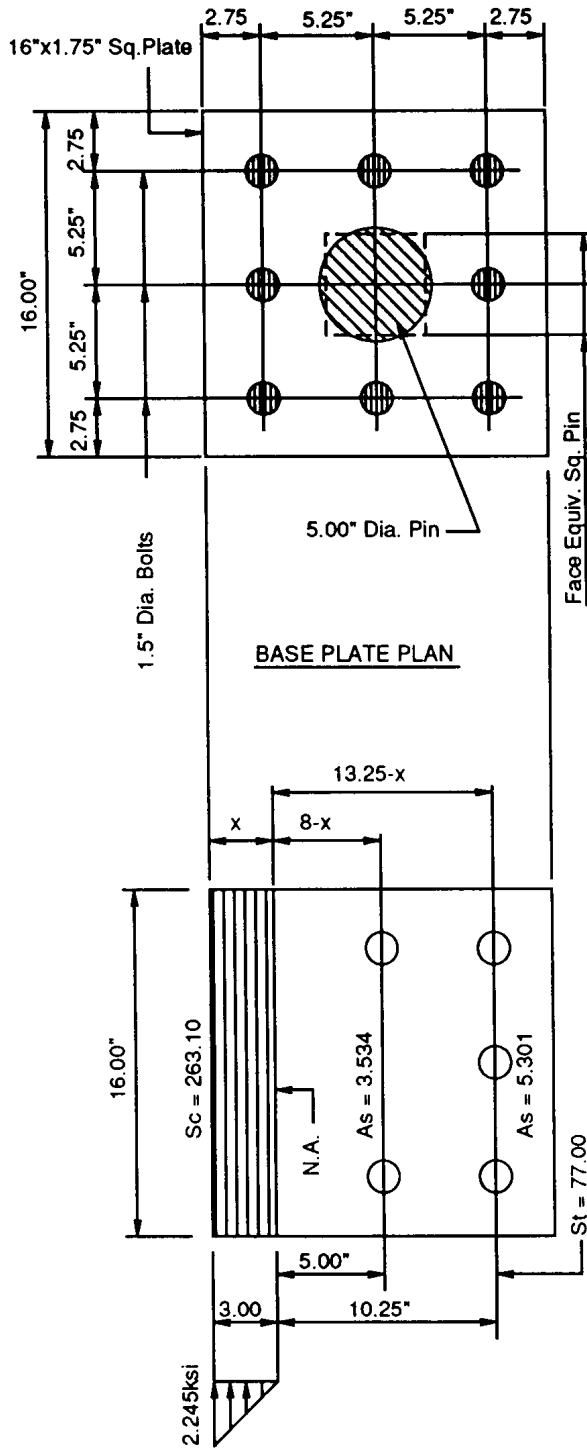
$P = \sqrt{Px^2 + Py^2} = 55.79k$

Moment at Bottom of Plate;

M=55.79x9.6+12.5x4.41=590.71k-in

(neglect 12.5k vertical load)

Calculate Section Properties

At Bottom Of Plate.

$8x^2 = 3.534(8-x) + 5.301(13.25-x)$

$8x^2 + 8.835x - 98.51025 = 0$

x = 3.00"
Bolt Mat'l. F593, Alloy 316, Fy=30ksi

$I_{N.A.} = 16(3)^3/3 + 3.534(5)^2$

$+5.301(10.25)^2 = 789.29in^4$

Sc = 789.29/3 = 263.10

St = 789.29/10.25 = 77.00

$f_c = 590.71/263.10 = 2.245ksi$

$f_t = 590.71/77.00 = 7.672ksi$

$F_t = 1.11x0.6x30 = 20ksi$

$F_v = 1.11x0.4x30 = 13.33ksi$

$f_v = 55.79/(8x1.767) = 3.95ksi$

USE 8-1.5" DIA. BOLTS F593

Moment in Plate at Edge of

Equiv. Square Pin =

0.5x2.245x3.00x4.78=16.10k-in

$S_{PL} = 1(1.75)^2/6 = 0.510$

Mat'l. ASTM A276, Fy=55ksi

1.11Fb = 1.11x0.75x55 =45.83ksi

$f_b = 16.10/0.510 = 31.57ksi < 45.83ksi$

USE 16" SQ. x1.75" A276 PLATE

C-11

EMBEDDED PLATE & BOLTS: (Bolt Mat'l. F593, Alloy 316. Plate A36)

26"SQ.x1.5" EMBED.PLATE

2.75" 6.75" 7.00" 6.75" 2.75"

2.75"

6.75"

7.00"

6.75"

2.75"

26"

1.5" dia. bolts

16" Sq.

Base Plate

5"

5"

5" 16" 5"

PLAN

$f_c = 0.2884\text{ksi}$

0.1387ksi

$f_t = 0.4079\text{ksi}$

x=9.63 13.62"

STRESS DIAGRAM
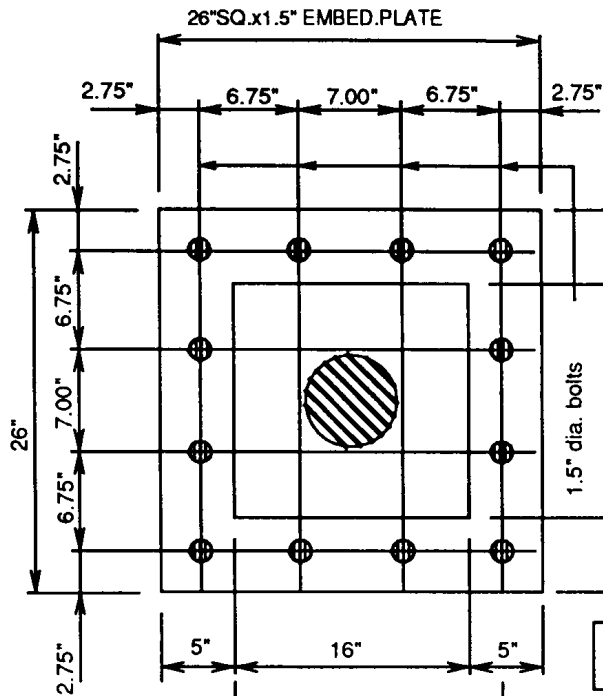
Properties of Transformed Section At Base Of Embed. Plate:

$n = E_s/E_c = 10$

$13x^2 = 35.34(16.5-x) + 70.68(23.25-x)$

$13x^2 + 106.02x - 2226.42 = 0$

$x = 9.63"$

$I_{N.A.} = 22519.21\text{in}^4$

$S_c = 2338.44, S_t = 1653.39$

$M = 55.79 \times 11.1 + 12.5 \times 4.41 = 674.39\text{k-in}$

$f_c = M/S_c = 0.2884\text{ksi}$

$f_t = M/S_t = 0.4079\text{ksi}$

Bolt Tension $= n f_t = 4.079\text{ksi}$

$F_t = 20\text{ksi}, F_v = 13.33\text{ksi}$

USE 12-1.5" DIA. F593 BOLTS

Stress At Edge Of Base Plate:

$S_{PL} = (1.5)^2/6 = 0.375$

$M = 0.5(0.2884+0.1387)(5)(2.79) = 2.98\text{k-in}$

$f_b = 2.98/0.375 = 7.95\text{ksi}$

Stress At Edge Of Equiv. Pin:

$M = 0.5(0.2884)(9.63)(8.57) = 11.90\text{k-in}$

Combined Section Modulus for Two Plates:

$S = 0.375 + 0.510 = 0.885$

$f_b = M/S = 11.90/0.885 = 13.45\text{ksi}$
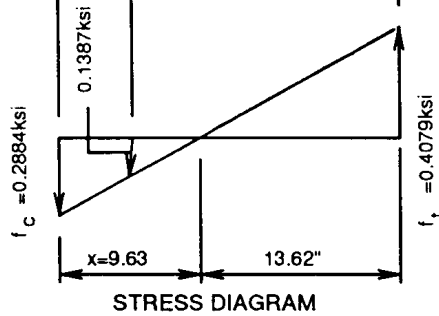
USE 26" SQ. x 1.5" A36 PLATE

See Page C-13 For Bending Stress In Embedded Bolts & Bolt

Bearing Stress On Concrete.

BOLTS IN CONCRETE

Using Maximum Allowable Concrete Bearing Stress, Find The

Minimum Required Bolt Embedment:

f_c'=3ksi (28 day concrete compressive strength)
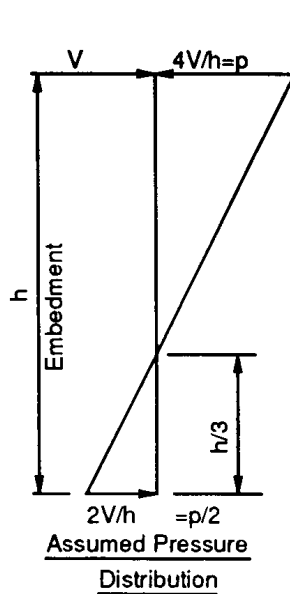
h=Min. Embed.=4V/p

F_p=0.35f_c'=1.05ksi From Sect. J9 Of AISC (1989)

d = Bolt Dia. =1.5" F_p(for Case I2)=1.11x1.05=1.167ksi

p = dF_p = 1.5x1.167 = 1.75k/in

V = 55.79/12 =4.65k/Bolt

h = 4x4.65/1.75 = 10.63" = Min. Embed. Req'd.

Actual Embedment = 20">10.63" o.k.

Diagram labels: V, 4V/h=p, h, Embedment, h/3, 2V/h, =p/2, Assumed Pressure Distribution

Calculate Flexural Stress In Bolt:

M = Max. Moment = 4Vh/27 = 4x4.65x10.63/27 = 7.323k-in

S = Bolt Sect. Mod. = 0.3313

1.11(0.75F_y) = 1.11x0.75x30 = 25ksi = F_b

f_b= M/S = 7.323/0.3313 = 22.10ksi<25ksi o.k.

CALCULATE BOND STRESS DUE TO BOLT TENSION:

f_t= Bolt Tensile Stress = 4.079ksi (from page C-12)
T=Bolt Tension=4.079x1.767=7.21 k/bolt
L=Embedment=20", C=Bolt Circumference=4.71"
U=Allowable Bond Stress Concrete To Plain Bar
U=0.09ksi (3000psi Concrete)
u=Actual Bond Stress=T/CL
u=7.21/(20x4.71)=0.0765ksi < U=0.09ksi

> BOND STRESS ALONE IS SUFFICIENT TO RESIST
> ANCHOR BOLT TENSION; HOWEVER, BOLTS SHALL
> HAVE 90 DEGREE HOOKS WITH 6" LEGS.

BUSHING DESIGN : (Mat'l. ASTM B22)

F_p=Max.Allowable Avg. Brg. Stress=1.11x3.00=3.33ksi

L_{min}=P/(F_pxD)=55.79/(3.33x5)=3.35"   USE L=6"

See Page B-15 For Calculation Of Actual Max. Brg. Stress.
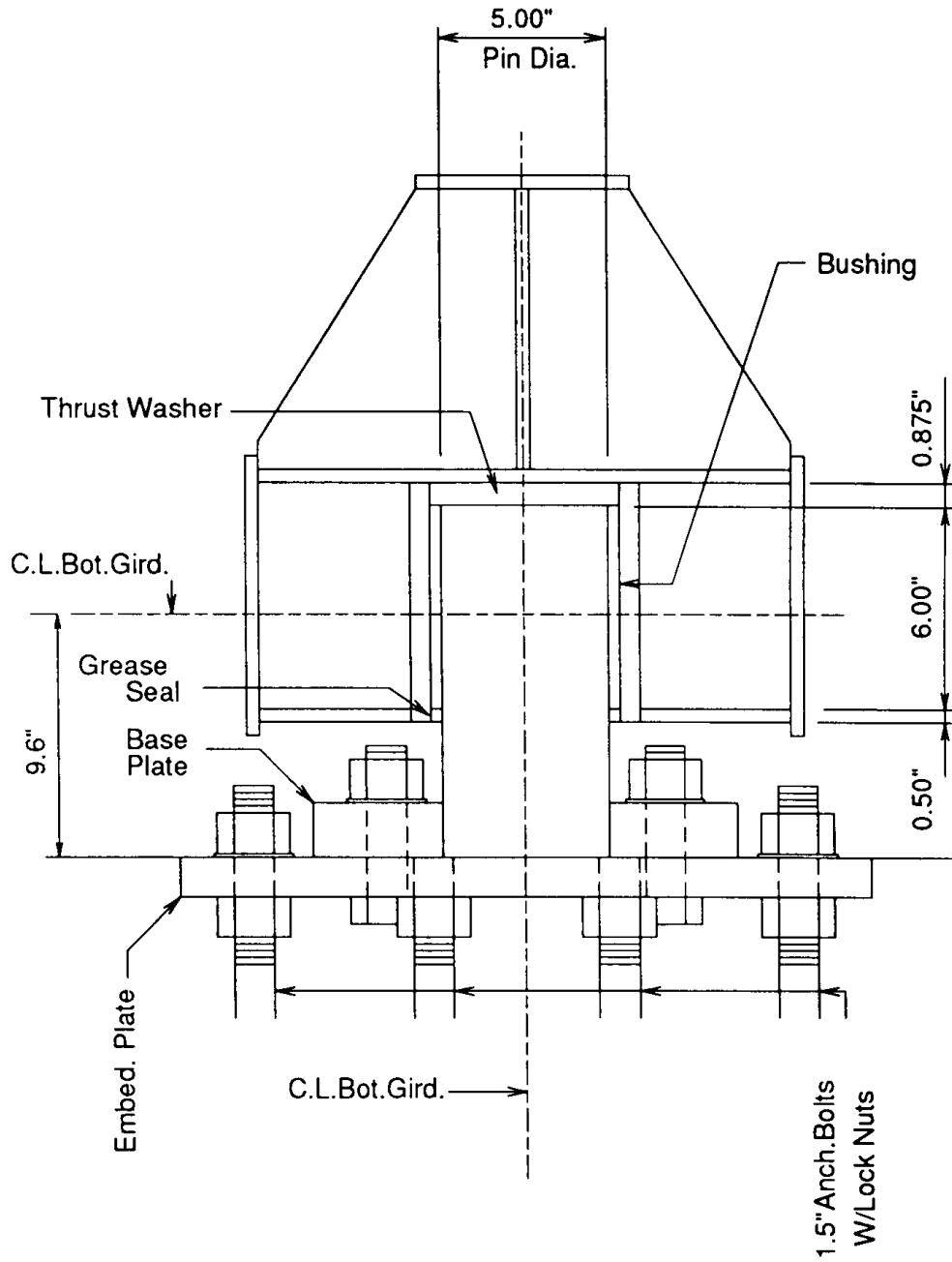
5.00"
Pin Dia.

Bushing

Thrust Washer

0.875"

C.L.Bot.Gird.

6.00"

Grease
Seal

9.6"

Base
Plate

0.50"

Embed. Plate

C.L.Bot.Gird.

1.5"Anch.Bolts
W/Lock Nuts

SECTION THRU BOTTOM HINGE

CALCULATION FOR ACTUAL MAXIMUM BEARING PRESSURE-BOTTOM PIN

L = Bushing Length = 6", Nominal Pin Diameter = 5"

R1 = Min. Radius Of Pin = 2.4985"

R2 = Max. Inside Radius Of Bushing = 2.5010"

E1 = Modulus Of Elasticity Of Pin = 29000ksi

E2 = Modulus Of Elasticity Of Bushing = 15000ksi

P1 = 55.79/6 = 9.30k/in

$$f_{p\ max.} = 0.591 \sqrt{\frac{P1\ E1\ E2\ (R2-R1)}{(E1+E2)(R1R2)}}$$

$$f_{p\ max.} = 0.591 \sqrt{\frac{9.3 \times 435 \times 10^6 \times 0.0025}{44 \times 10^3 \times 6.24875}} = 3.584ksi$$

A Maximum Bearing Stress Equal To Or Less Than The Yield
Strength Of The Material Is Allowable.

C-15

TOP HINGE:
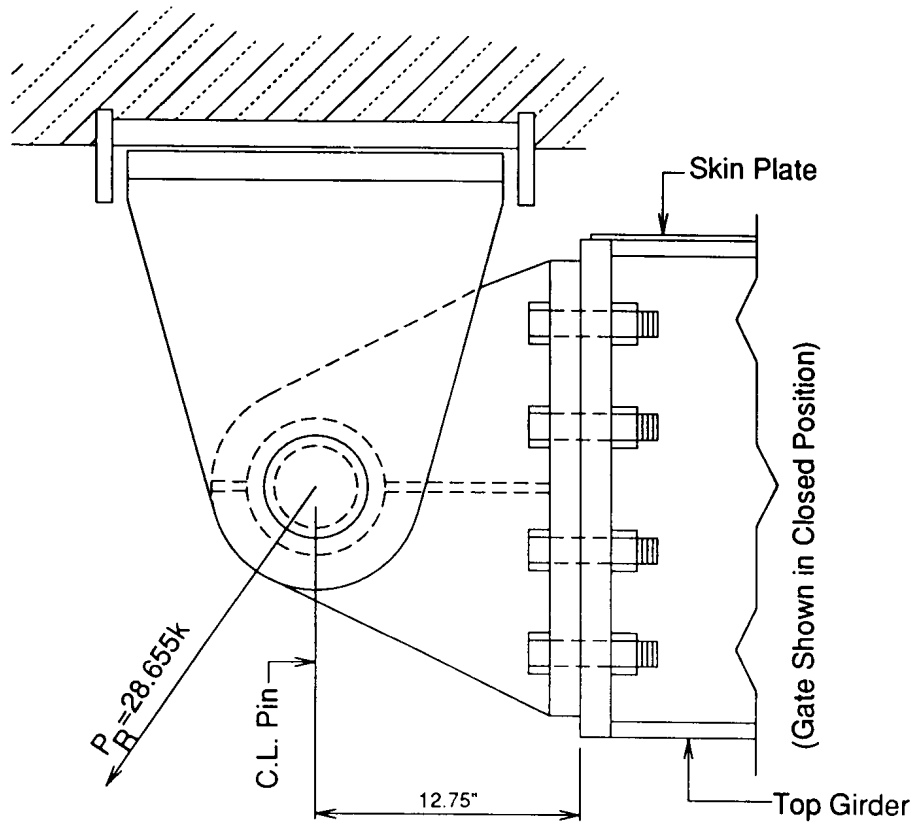
PLAN
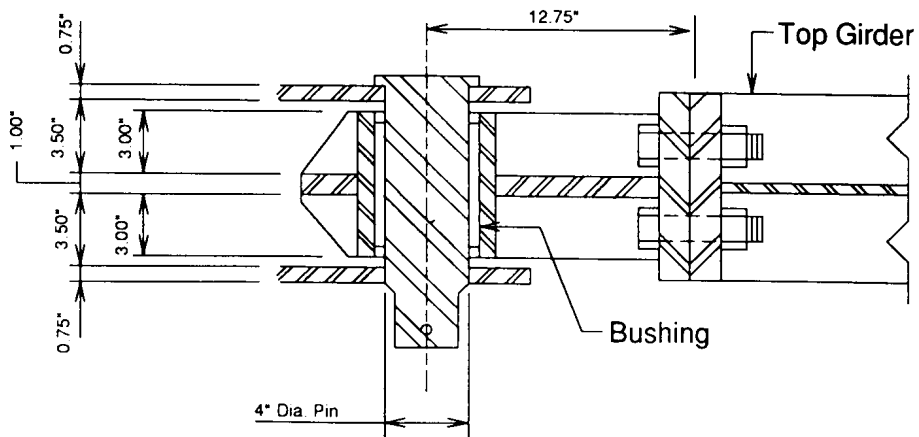
SECTION THRU HINGE

<u>TOP PIN</u> (Mat'l. ASTM A276, Fy = 55ksi)

P_R = 28.655k, V = 28.655/2 = 14.33k (Dbl. Shear)

M = 28.655x8.75/4 = 62.68k-in

$A = 3.1416(4)^2/4 = 12.57$, $S = 3.1416(4)^3/32 = 6.28$

F_b = 1.11(0.75F_y)=45.83ksi

F_V = 1.11(0.4F_y) = 24.44ksi

f_V=14.33/12.57 = 1.14ksi < 24.44ksi o.k.

f_b = 62.68/6.28 =9.98ksi < 45.83ksi   o.k.

<u>TOP BUSHING</u> (Mat'l. ASTM B22)

P_R = 28.655k, Bushing Length = 6.00"

Inside Dia. = 4.00"

F_p = Avg. Allowable Brg. Stress =1.11x3.00 = 3.33ksi

f_p= 28.655/(4x6) = 1.194ksi < 3.33ksi o.k.

CHECK MAX. BEARING STRESS:

R1 = 1.997", R2 = 2.001"

E1 = 29000ksi, E2 = 15000ksi

P1 = 28.655/6 = 4.78k/in

(Max) f_p= 0.591 $\sqrt{\dfrac{4.78 \times 29 \times 15 \times 10^6 (2.001-1.997)}{(29+15)(10^3)(2.001 \times 1.997)}}$ = 4.065ksi

A Maximum Bearing Stress Equal To Or Less Than The Yield
Strength Of The Material Is Allowable.

Check Bottom Girder Design With Axial Load & Bending Due To Diagonal Included.

P = Axial Load From Diagonal = 7.808 k (See page C-8)
e = Eccentricity = 10.674", Pe = 83.34 k-in
M(Hs) = Moment Due To Hydrostatic Load = 4992 k-in (See page C-7)
M = Pe + M(Hs) = 83.34 + 4992 = 5075.34 k-in

Properties of W24×84:

$$A = 24.7, \ S = 196, \ r_x = 9.79, \ \frac{b_f}{2t_f} = 5.9, \ \frac{d}{t_w} = 51.3$$

Member is Compact (Table B5.1) $L_x = 379", \ L_b = 66"$

$$\frac{KL_x}{r_x} = 39, \ f_a = \frac{7.808}{24.7} = 0.316 \ \text{ksi}, \ F_a(AISC) = 19.27 \ \text{ksi}$$

$$F_a = 1.11 F_a(AISC) = 21.41 \ \text{ksi}, \ F_b = \frac{M}{S} = 25.89 \ \text{ksi}$$

$$\frac{76 b_f}{\sqrt{F_y}} = 114'' > L_b = 66'' , \ F_b(AISC) = 24 \ \text{ksi}, \ F_b = 1.11 \times 24 = 26.67 \ \text{ksi}$$

$$\frac{f_a}{F_a} < 0.15, \ \frac{f_a}{F_a} + \frac{f_b}{F_b} = \ < 1.00 \ (H1-3)$$

$$\frac{0.316}{21.41} + \frac{25.89}{26.67} = 0.986 < 1.00$$

W24×84 Satisfies Axial Load & Flexural Requirements.

Appendix D
Miter Gate Design Example

The opening in this example is 36 ft wide by 11.125 ft high.

Load Cases:

In accordance with EM 1110-2-2502, consideration was given to the following cases applicable to inland floodwalls.

Case I1, Design Flood Loading. Gate is mitered; water on the unprotected side is at the design flood elevation; water is at or below sill on protected side. Design stresses shall not be greater than 5/6 of stresses allowed in AISC (1989).

Case I2, Maximum Flood Loading. Same as Case I1 except that water level is to top of gate on unprotected side. Design stresses shall not be greater than 1.11 times the stresses allowed in AISC (1989).

Case I3, Earthquake Loading. Water is at usual level (nonflood condition) on unprotected side; earthquake-induced forces are acting. (Note: This case is applicable to support structures only.)

Case I4, Short-Duration Loading. Gate is either open or mitered or in between and is subjected to construction and/or wind loads. Design stresses shall not be greater than 1.11 times the stresses allowed in AISC (1989).

In this example, cases I1 and I3 are not significant and skin plate, intercostals, and girders are designed for Case I2. Case I4 is applicable to the design of diagonals and latching devices.

The skin plate is designed as a fixed end member spanning between intercostals. The hydrostatic pressure 6 ft above the flange of the bottom girder is used as a uniform load. In order for the design to meet the limitations of the flat plate theory, deflection is limited to 0.4 of plate thickness.

The intercostals are designed as simple beams spanning between girders.

Girders are designed as elements of a three-hinged arch. They are designed for thrusts and moments induced by diagonal tensions as well as for hydrostatic pressure.

1/2 HORIZONTAL SECTION THRU GATE

Water Surface
Max. Flood

Top of Gate

Top Girder

0.375'

Water
Surface
Design
Flood

3.00'

3.33'

Gird. Flg. Brace
Plate @ 6'-0"

Skin Plate

Intercostal
Flg. Brace

Wet Side

3.34'

10.00'

11.125'

Dry Side

Intercostals

3.33'

Gird. Flg. Brace
Plate @ 6'-0"

Continuous
"J" Seal

Sill
Sealing
Surface

0.75

Bottom Girder
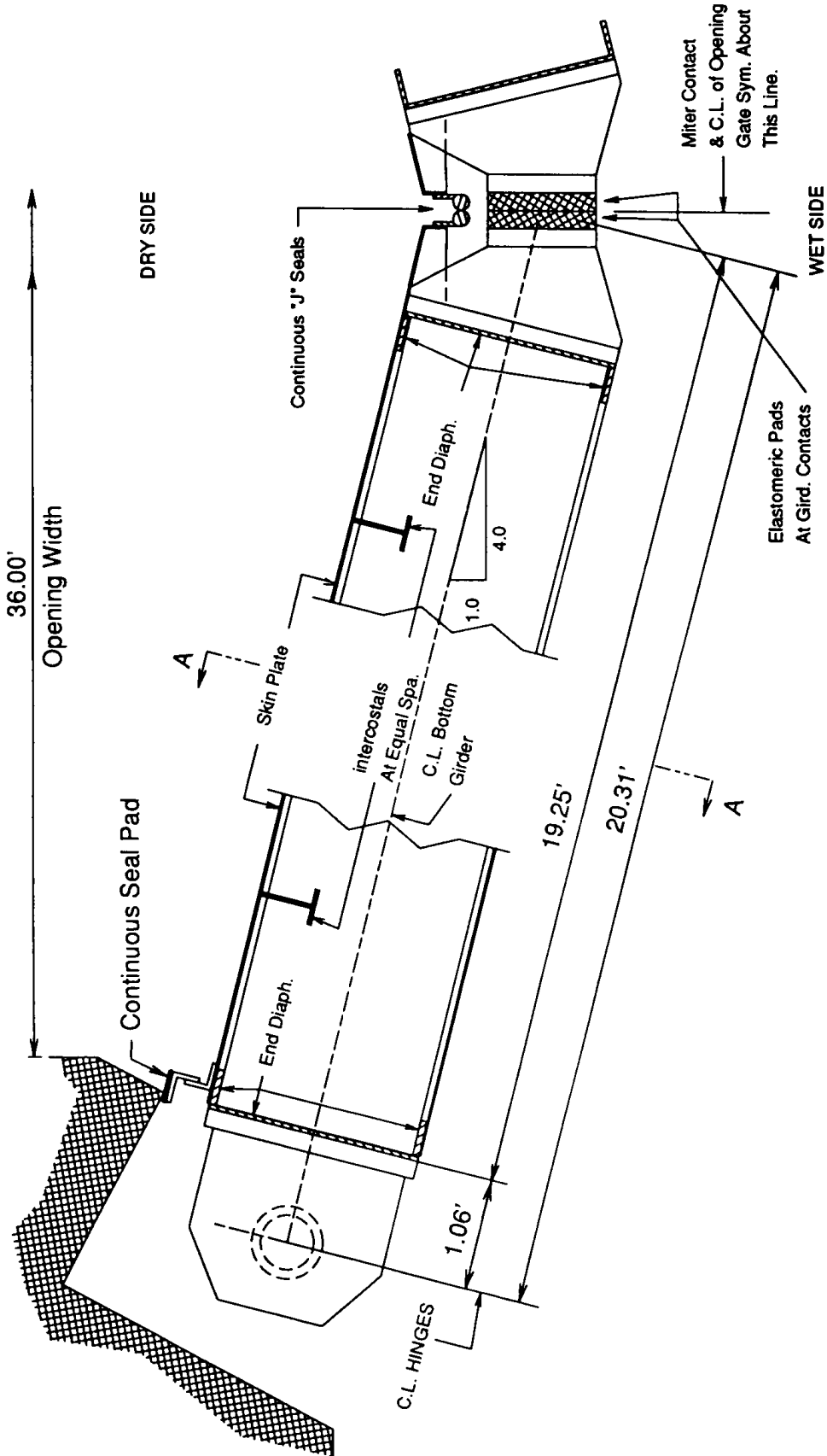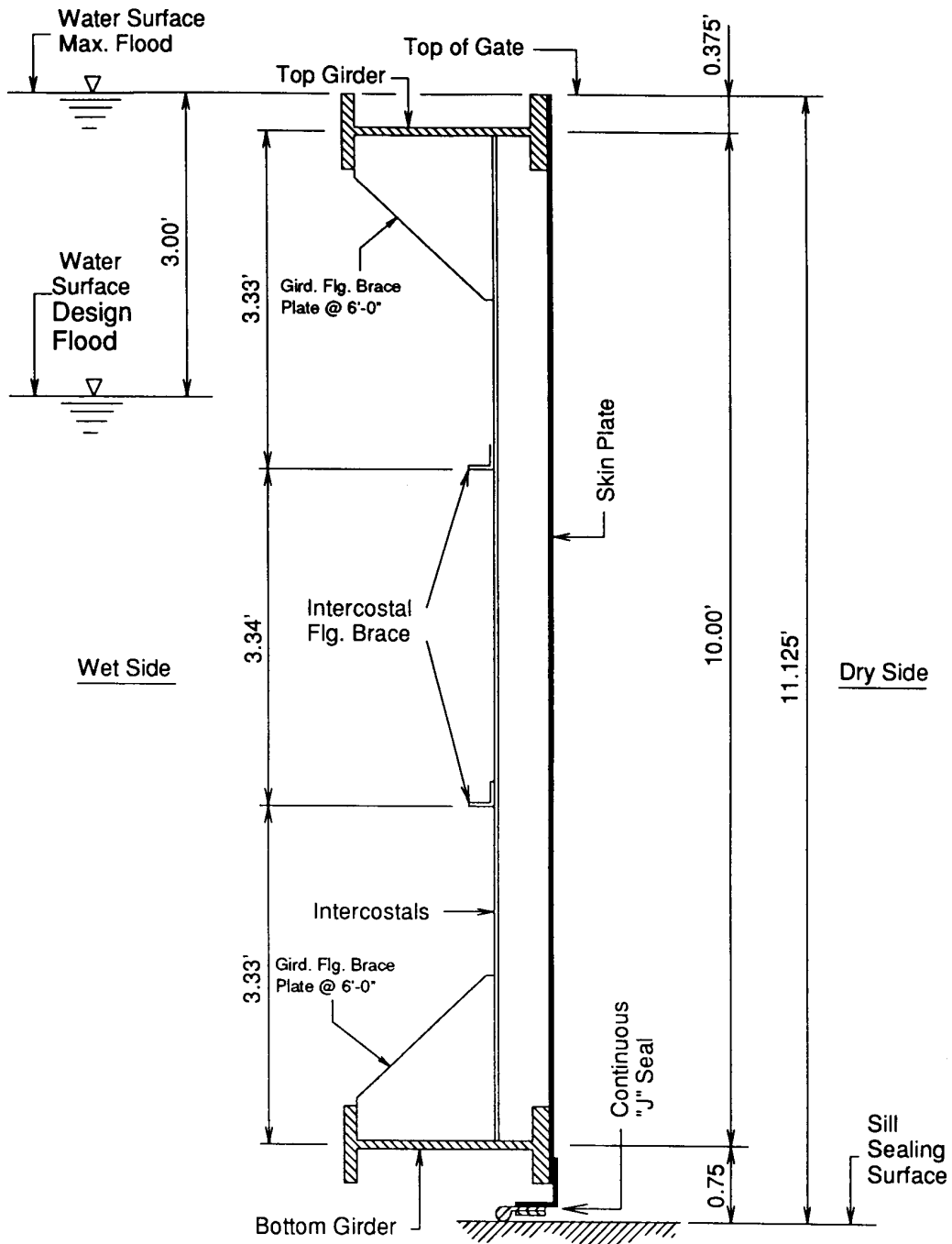
SECTION A—A

Skin Plate Design; Load Case I2:

Assuming 9" wide girder flanges, the hydrostatic pressure for skin plate design is:

p=0.0625(11.125-0.75-0.375-.5)=0.5983ksf

p=0.004124ksi, b=intercostal spacing=24"

$M=pb^2/12=0.004124(24)^2/12=0.1980$k-in

$F_b=1.11(0.75F_y) = 1.11 \times 0.75 \times 36 = 30$ksi

$t_{min-stress}=(pb^2/2F_b)^{1/2}$

$t_{min-stress}=[0.004124(24)^2/(2 \times 30)]^{1/2}=0.1990$"

Defl.$=pb^4/384EI$; E=29000; $I=t^3/12$; Defl.$=0.4t$

$0.4t=12pb^4/384Et^3$, $t^4=pb^4/12.8E$

$t_{min-defl.}=[0.004124(24)^4/(12.8 \times 29000)]^{1/4}$

$\underline{t_{min-defl.}=0.2462"}$

> USE: 1/4" Skin Plate.

Intercostal Design; Load Case I2:

Eq. & Table No's. In Parentheses Are From AISC (1989). Load, shear, and moment diagrams for intercostals are shown on page D-5.

M=4.27x2'=8.54k-ft=102.48k-in

The trial section for intercostals is shown on page D-6.

$S=5.47$in^3, $L_b=40$"(dist. between flg. braces)

$b_f/2t_f=3.96/(2 \times 0.21)=9.43$

$65/(F_y)^{1/2}=65/6=10.83>9.43$, compact (Table B5.1)
$L_c =76b_f /(F_y)^{1/2}=76(3.96)/6 = 50.16">40"$ (F1-2)

$F_b=1.11(2/3)F_y=1.11(2/3)(36) = 26.67$ksi
$f_b=M/S=102.48/5.47=18.73ksi<F_b=26.67$ksi

> USE: WT 5x6 for Intercostals.

Water Surface
& Top of Gate

0.375'

1.142 k/ft

0.047 ksf

0.75'

0.0625 ksf/ft

9.25'

Intercostals

M(max) = 4.27 k-ft/ft

-1.967 k/ft

0.625 ksf

1.125'

Seal
Surface

0.375'
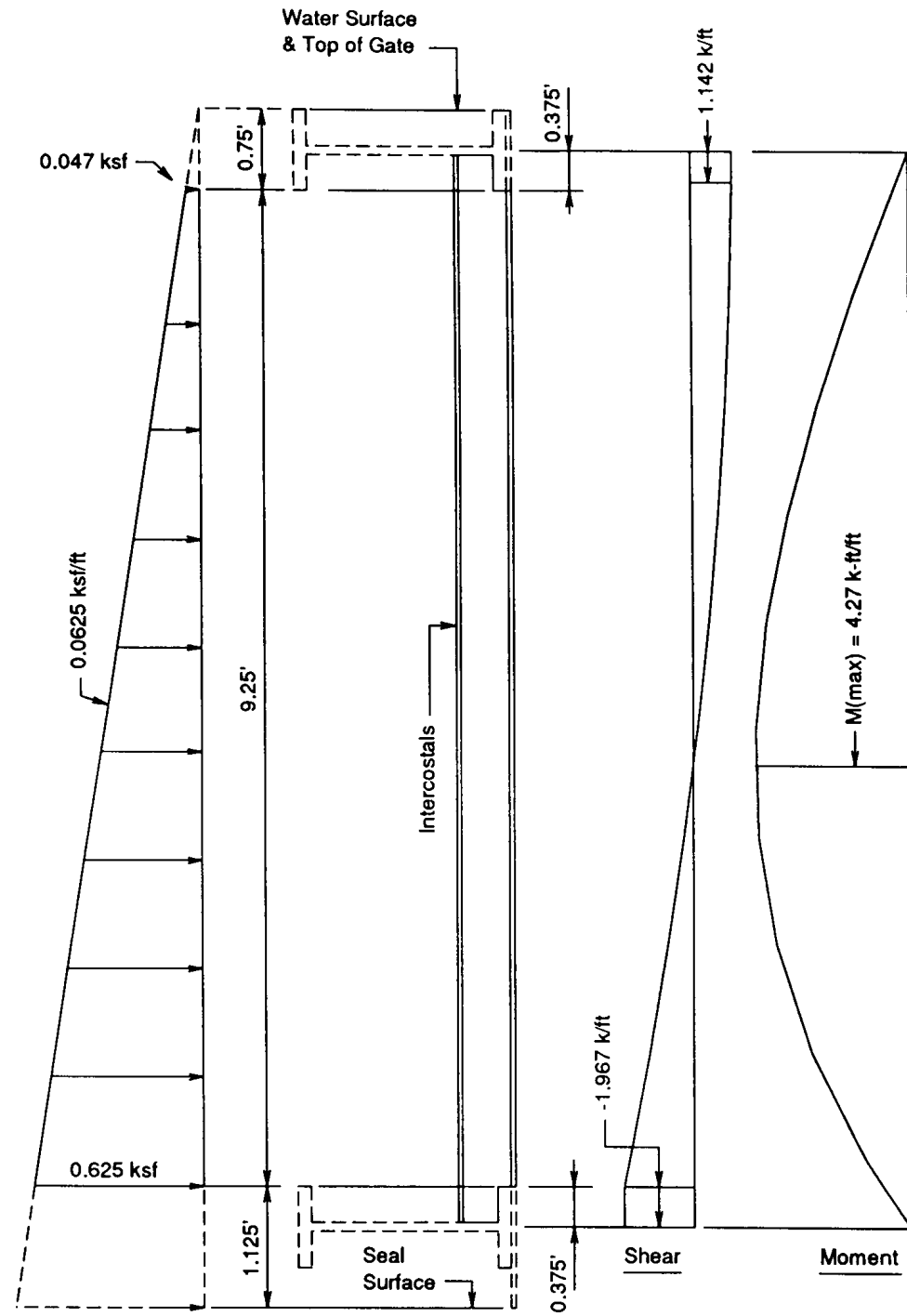
Shear

Moment

Hydrostatic Load

LOAD, SHEAR, & MOMENT DIAGRAMS
FOR
INTERCOSTALS
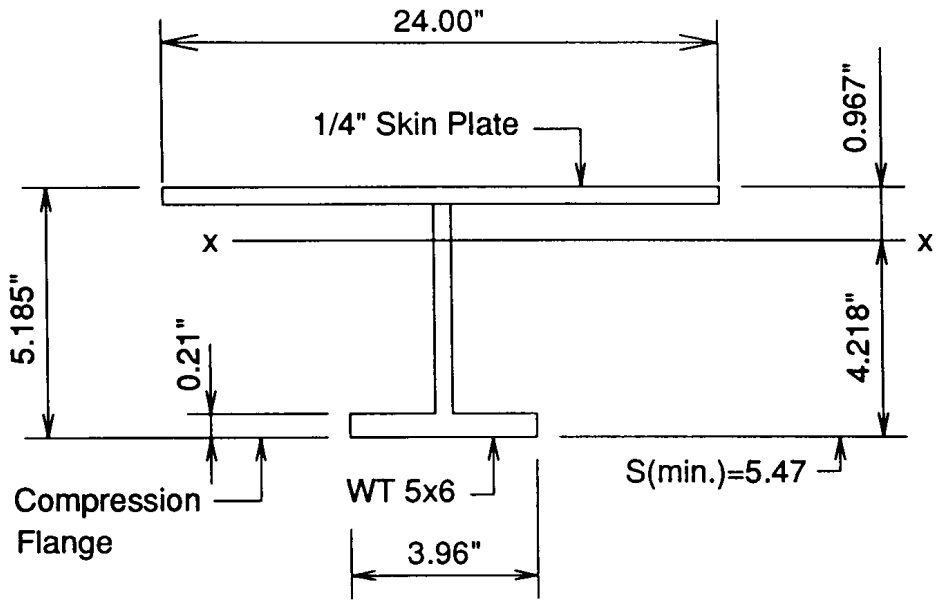
INTERCOSTAL SECTION

CALCULATION FOR DIAGONAL LOADS

Methods For Determining The Shear Center And The Torsional Stiffness

Of The Gate Are Presented In EM 1110-2-2703.
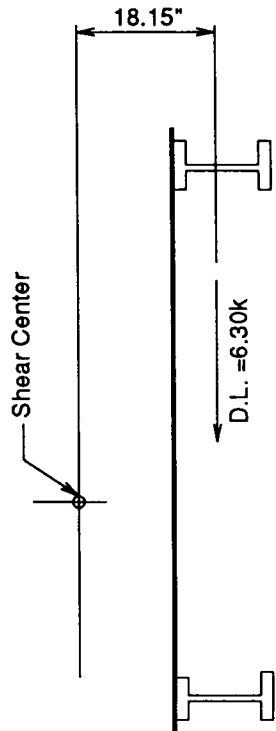
T=Torsion=6.30x18.15=114.35k-in

Z=Distance From Hinges To C.G. Of D.L.

Z=123"

$TZ=14065\text{k-in}^2$

A=Area Of Diagonal=0.79

A'=1/8 Of Sum Of Girder & Diaph. Areas

A' = 8.05

L=Length Of Diagonal=253.24"

Qo=Torsional Stiffness Of Gate Leaf=1222

t=Dist. From C.L. Of Skin To C.G. Of Diagonal

t =24.2"

Ro=2wt/vL=2x223x24.2/(243.72x253.24)=0.175

R=A' Ro/(A+A')

R=8.05x0.175/(0.79+8.05)

R=0.159

Q=RRoEAhv/L=Torsional Stiff. Diagonal

Q=0.159x0.175x29000x0.79x120x243.72/253.24=73620

(Diagram labels: 18.15", Shear Center, D.L. =6.30k, End View)

QDp + QDn = TZ Where Dp is Prestress Delection For Positive Diagonal and

Dn is Prestress Deflection For Negative Diagonal.

Positive Diagonal Extends From Top Girder At Hinge End To Bottom Girder

At Miter End.

Let Dn = 0.40", QDn = 73620x0.40 = 29448

73620 Dp = 14065 + 29448 = 43513, Dp = 43513/73620 = 0.59"

Sp = RDpE/L = 0.159x0.59x29000/253.24 = 10.74ksi

Sn = RDnE/L = 0.159x0.40x29000/253.24 = 7.28ksi

Tension In Positive Diagonal = 0.79x10.74 = 8.48k

Tension In Negative Diagonal = 0.79x7.28 = 5.75k
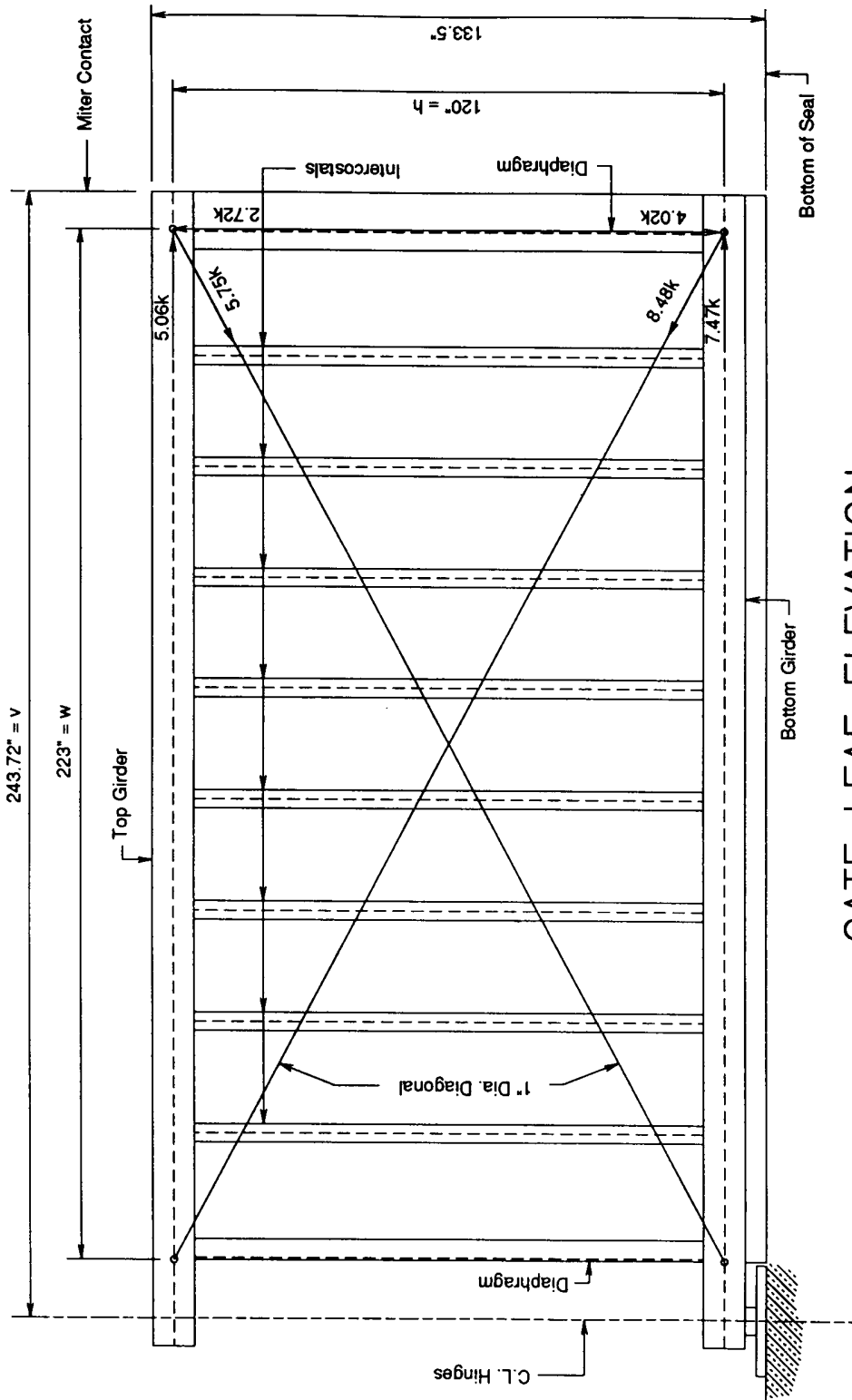
GATE LEAF ELEVATION
SHOWING DIAGONAL LOADS

0.375'

Top Girder

1.144 k/ft

0.0625 ksf/ft

Skin Plate

Girder Loading

10.00'

11.125'

Bottom Girder

2.724 k/ft

0.75'

0.6953 ksf
Hydrostatic Load
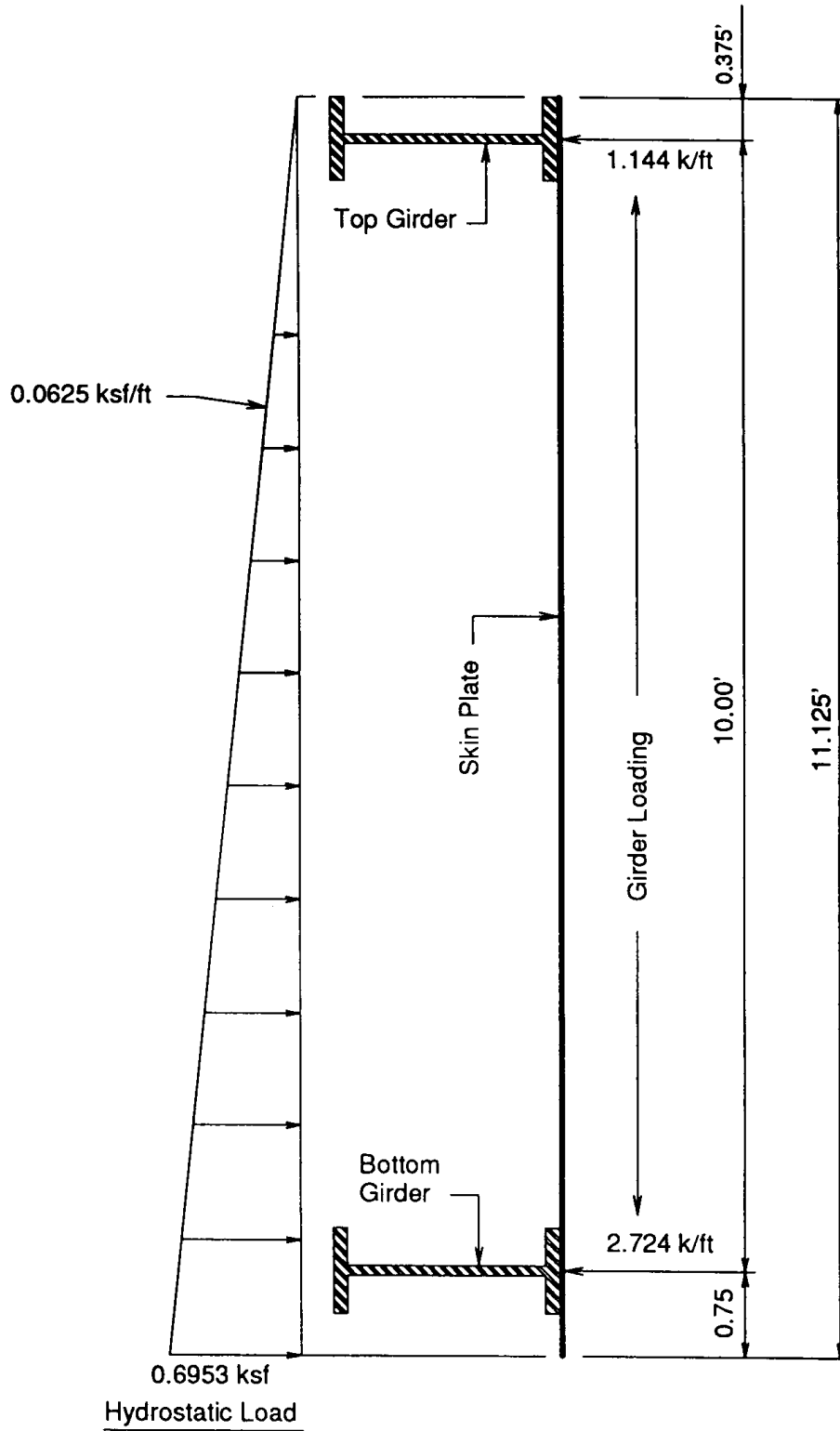
GIRDER LOADING

HYDROSTATIC LOADING & REACTIONS FOR BOTTOM GIRDER:
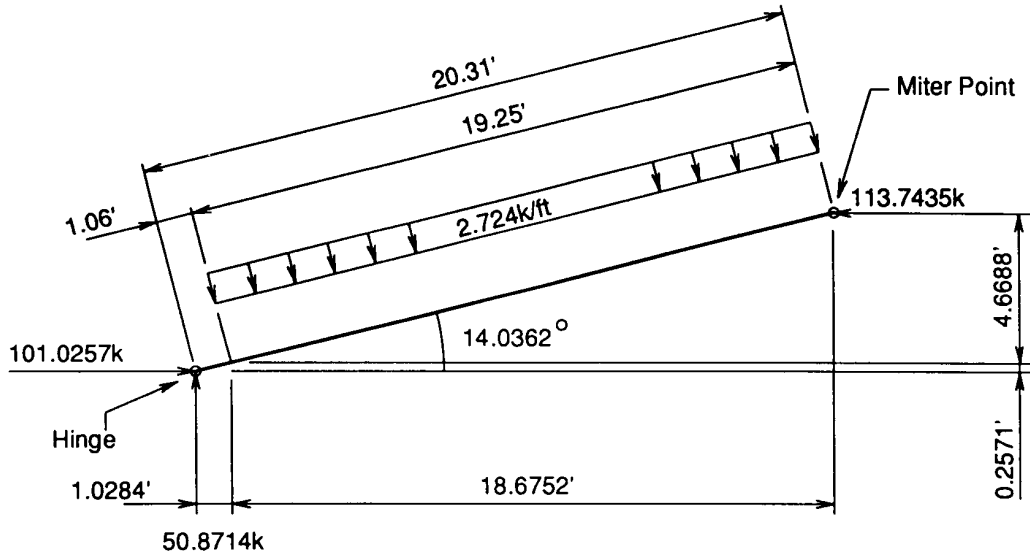
Free Body Of Bottom Girder

Resolve Reactions At Hinge Into Components Normal
And Parallel To Girder Centerline:

V = Normal Component, T = Parallel Component

V = 50.8714xCos(14.0362) - 101.0257xSin(14.0362) =24.850k

T = 50.8714xSin(14.0362) + 101.0257xCos(14.0362) = 110.347k

Resolve Reaction At Miter Point Into Components Normal
And Parallel To Girder Centerline:

V = 113.7435xSin(14.0362) = 27.587k

T = 113.7435xCos(14.0362) = 110.347k

2.724 k/ft

1.06'

19.25'

112.757k

End Diaph.

110.347k

Hinge

5.06k

1.024'

7.47k

Miter
Contact

Diag. Load

24.9715k

27.4655k

27.4655k

SHEAR

24.9715k

MOMENT

M(max)=146.11k-ft
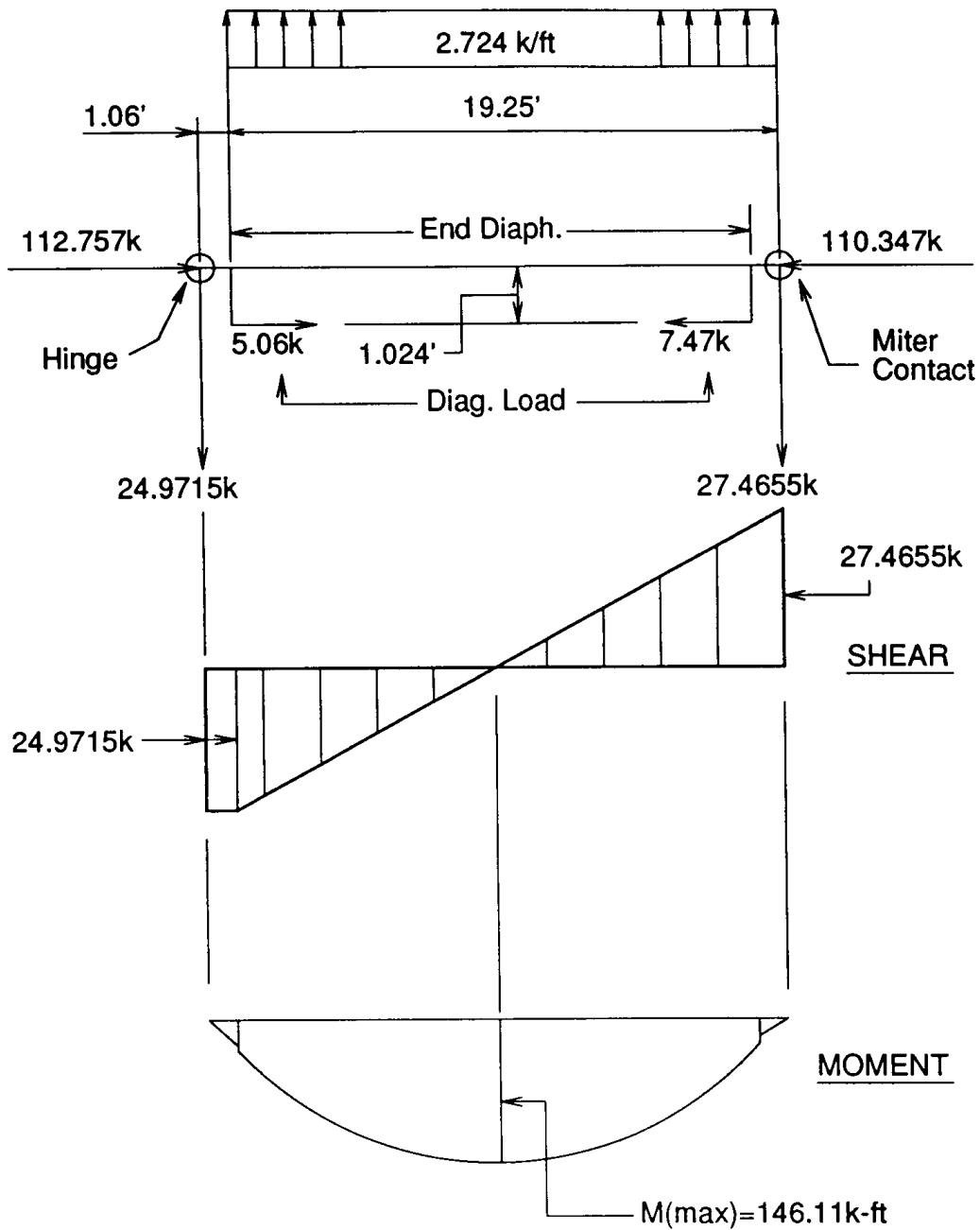
LOAD, REACTIONS, SHEAR, & MOMENT FOR BOTTOM GIRDER

BOTTOM GIRDER DESIGN

Material: ASTM A36

Equation & Table Numbers In Parentheses Are From Spec. In AISC (1989).

Trial Section, W 24x55.

$A=16.2$, $S_x=114$, $r_x=9.11$, $r_y=1.34$, $r_T=1.68$, $d/A_f=6.66$

$b_f=7"$, $t_f=0.505"$, $b_f/2t_f=6.9$, $d=23.57"$, $h=22.56"$

$t_w=0.395"$, $d/t_w=59.7$, $h/t=57.11$

$L_x=20.31'=244"$, $L_y=24"$, $L_b=72"$

$P=112.757+5.06=117.817k$, $M=146.11k\text{-ft}=1753.32k\text{-in}$

Allowable Stresses:

$65/(F_y)^{1/2}=65/6=10.83>6.9$ (Table B5.1)

$L_c=20000/(6.66\times36)=83.42>L_b=72"$

$640/(F_y)^{1/2}=640/6=106.67>59.7$ (Table B5.1)

W 24x55 is compact for flexure, $F_b=1.11(2/3)F_y=26.67ksi$

$257/(F_y)^{1/2}=257/6=42.83<59.7$ (Table B5.1)

Web is a slender element under uniform compression.

$h_e=[253t_w/(f)^{1/2}][1-44.3t_w/h(f)^{1/2}]$ (A-B5-8)

$f_a=P/A=117.817/16.2=7.27ksi$, $f_b=M/S=1753.32/114=15.38ksi$

$f=7.27+15.38=22.65ksi$, $(f)^{1/2}=4.76$

$h_e=[253\times0.395/4.76][1-44.3/(57.11\times4.76)]=17.57"$

$A_e=16.2-(22.56-17.57)(0.395)=14.23$

$Q=A_e/A=14.23/16.2=0.8784$

$L_x/r_x=244/9.11=27$, $L_y/r_y=24/1.34=18$

$F_a=1.11\times20.15=22.39ksi$ (E2-1)

$F'_e=1.11\times204.84=227.6ksi$ (From Sect. H1 of AISC (1989)

$F_a=QF_a=0.8784\times22.39=19.67ksi$ (For Case I2)

Combined Stresses:

$f_a/F_a+Cmf_b/[1-f_a/F_e]F_b=<1.00'$ (H1-1)

$7.27/22.39 + 1\times15.38/[(1-7.27/227.6)(26.67)]=0.92<1.00$

USE W24x55

Also Use W 24x55 For Top Girder To Maintain 24" Depth So That Diagonals Lie In A Vertical Plane.

BOTTOM HINGE-PEDESTAL DESIGN

Result.Thrust = $\sqrt{(112.757)^2 + (24.9715)^2}$ = 115.49k
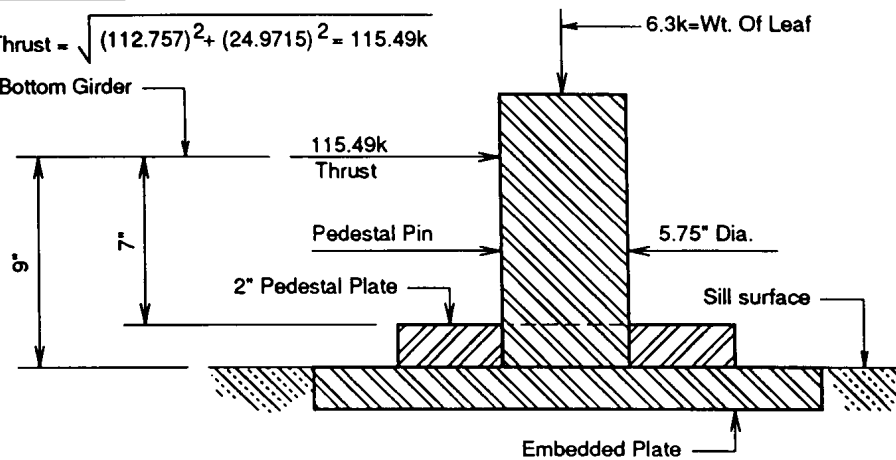
Section Thru Pedestal

Pin Mat'l.: ASTM A276

Pedestal Pin

F_y= 55 ksi. F_b= 1.11 (0.75F_y) = 45.83ksi

d = 5.75", A = 25.97 in^2, I = 53.66 in^4

S = 18.66 in^3, r = 1.44

L = 7", K = 2, KL/r =2x7/1.44 = 9.72

C_c=102 (Ref. 3.d.(1).)

FS = 5/3 + 3(KL/r)/(8xC_c) - [KL/rC_c]^3/8 = 1.70

Fa = [1-0.5(KL/Ccr)^2][Fy]/FS=32.17ksi (AISC 1989)

F_a = 1.11x32.17 = 35.74ksi

M = 115.49 x 7 = 808.43 k-in

f_a = 6.3/25.97 = 0.243 ksi

f_b = 808.43/18.66 = 43.32 ksi

f_a/F_a +f_b/F_b= 0.243/35.74+43.32/45.83=0.952<1.000

USE 5.75" Diameter Pin

PEDESTAL BASE PLATE

Material: Bolts F593, Alloy 316.   Plate ASTM A276.

16" SQ. PLATE

2.75" 5.25" 5.25" 2.75"

2.75"

5.25"

5.25"

2.75"

2" Thick

Equiv. Sq. Section for Round Pin

1.5" Dia. Bolts

2.55" 2.55"

PLAN

3.00" 5.00" 5.25"

16.00"

Sc=263.10 in^3

Neutral Axis

A=3.534 in^2

A=5.301 in^2

St=77.00 in^3

PROPERTIES OF SECTION
AT
BOTTOM OF PLATE
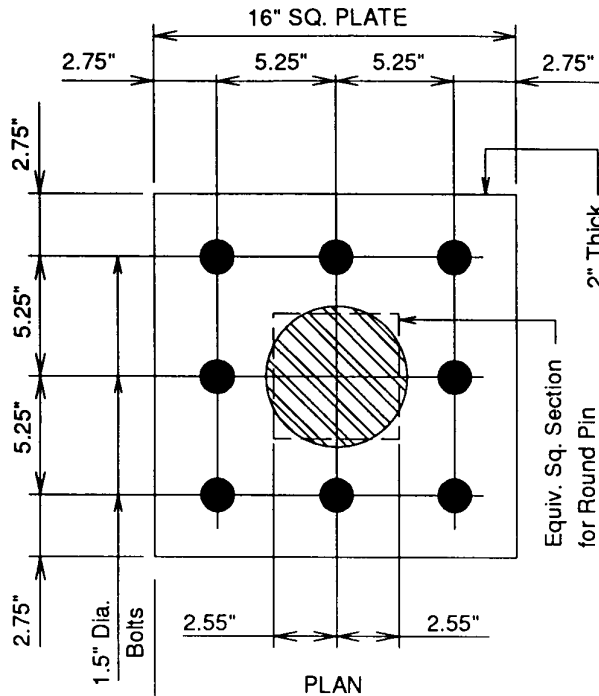
BOLTS:

$V = 115.49k$

$M = 115.49 \times 9 = 1039.41 k\text{-}in$

$f_c = M/Sc$

$f_c = 1039.41/263.10 = 3.95ksi$

$f_t = M/St$

$f_t = 1039.41/77 = 13.50ksi$

Bolt Area At Thread = 1.41

f_t (at thread) $= 1.767 \times 13.5/1.41 =$

$= 16.92ksi$

$F_y = 30ksi$

$F_t = 1.11 \times 0.6 \times 30 = 20ksi > 16.92ksi$

$f_v = 115.49/(8 \times 1.41) = 10.24ksi$

$F_v = 1.11 \times 0.4 \times 30 = 13.33ksi > 10.24ksi$

USE 8-1.50" DIA. BOLTS
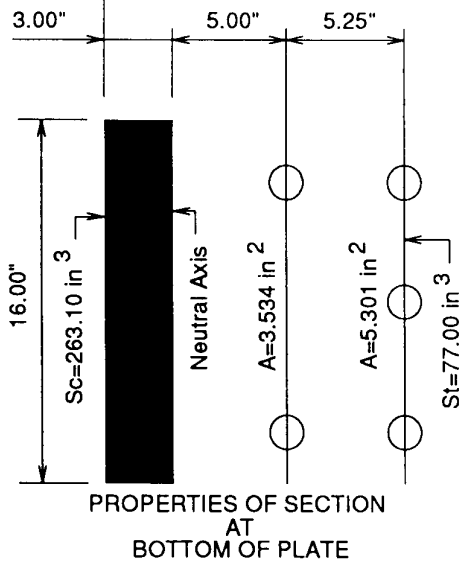
PLATE:

Moment in plate at edge of equiv. Sq. Pin.

$M = 0.5 \times 3.95 \times 3 \times 4.45 \times 16 = 421.86 k\text{-}in$

$S = 16(2)^2/6 = 10.67$

$f_b = M/S = 421.86/10.67 = 39.54ksi$

$F_y = 55ksi$

$F_b = 1.11 \times 0.75 \times 55 = 45.83ksi > 39.54ksi$

USE 16" SQUARE X 2" PLATE

EMBEDDED PLATE & BOLTS: (Bolt Mat'l. F593, Alloy 316. Plate A36)

PLAN

STRESS DIAGRAM
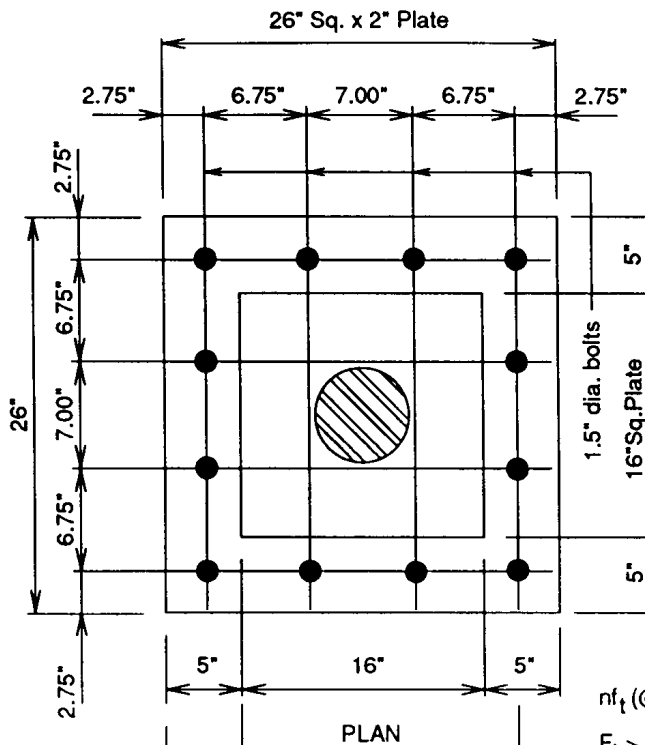
BOLTS:

Transformed Section, n = 10

$I_{N.A.}$=22519.21

Sc = 2338.44, St = 1653.39

V = 115.49k

M = 115.49x11 = 1270.39k-in

f_p= M/Sc = 0.543ksi

F_p = 1.11x0.35f'_c

F_p = 1.11x0.35x3 = 1.167ksi

$F_p > f_p$

f_t = M/St = 0.7684ksi

F_y = 30ksi

F_t = 1.11x0.6x30 = 20ksi

A(bolt)=1.767, A(@ th'ds.)=1.41

nf_t = 10x0.7684 =7.684ksi

nf_t(@th'ds)=1.767x7.684/1.41=9.63ksi

F_t > 9.63ksi

F_v= 1.11x0.4xF_y= 13.33ksi

f_v=115.49/(12x1.767)=5.45ksi<13.33ksi

USE 12-1.5" DIA. BOLTS

PLATE @ EDGE OF PED. PLATE:

M=0.5(0.543+0.261)(5)(2.79)(26)=

M = 145.80k-in , S=26(2)^2/6=17.33

f_b= M/S =8.41ksi

F_b = 1.11x0.67x36 = 26.67ksi > 8.41ksi

PLATE @ EDGE OF EQUIV.SQ. PIN:

M=0.5(0.543)(9.63)(7.24)(26)=

M = 492.16k-in

S(both plates)=10.67+17.33=28

f_b=M/S = 17.58ksi < 26.67ksi
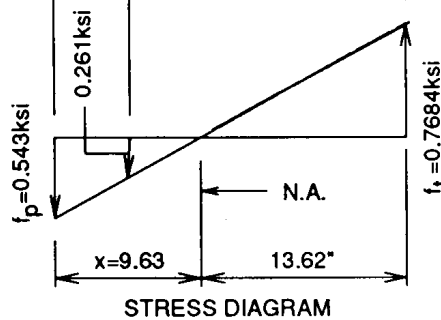
See Page D-16 For Bending Stress In Embedded Bolts & Bolt
Bearing Stress On Concrete.

BOLTS IN CONCRETE

Using Maximum Allowable Concrete Bearing Stress Find the

Minimum Required Bolt Embedment:

f'_c=3ksi (28 day concrete compressive strength)
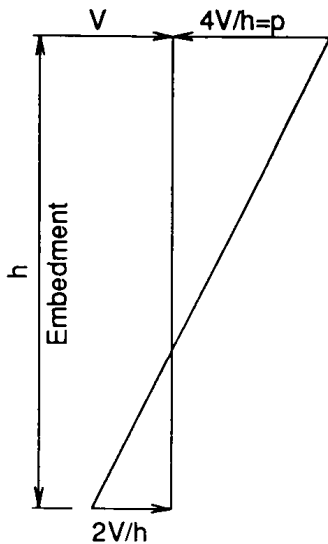
h=Min. Embed.=4V/p

F_p=0.35f'_c=1.05ksi From Sect. J9 Of AISC (1989).

d = Bolt Dia. =1.5" F_p (for Case I2)=1.11x1.05=1.167ksi

p = dF_p = 1.5x1.167 = 1.75k/in

Total Horizontal Load On Embedment=115.49k

Allowable Brg.Load On 2" Plate=1.167x26x2=60.68k

Shear Load On Embedded Bolts=115.49-60.68=54.81k

V=54.81/12=4.568k/bolt

h=4x4.568/1.75=10.44" (min req'd.)

Actual Embed.=17" > 10.44" o.k.

[Figure: diagram labeled V, 4V/h=p at top, h / Embedment on left side, 2V/h at bottom]

Calculate Flexural Stress In Bolt:

M=4Vh/27=4x4.568x10.44/27=7.065k-in

S = Bolt Sect. Mod. = 0.3313

1.11(0.75F_y) = 1.11x0.75x30 = 25ksi = F_b

f_b= M/S =7.065/0.3313 = 21.33ksi < 25ksi o.k.

BUSHING DESIGN   : (Mat'l. ASTM B22)

F_p =Max.Allowable Avg. Brg. Stress=1.11x3.00=3.33ksi

USE: L=7" , D=5.75" , P=115.49k , f_p=P/DL

f_p=115.49/(7x5.75)=2.869ksi<3.33ksi

See Page D-18 For Calculation Of Max. Bearing Stress.
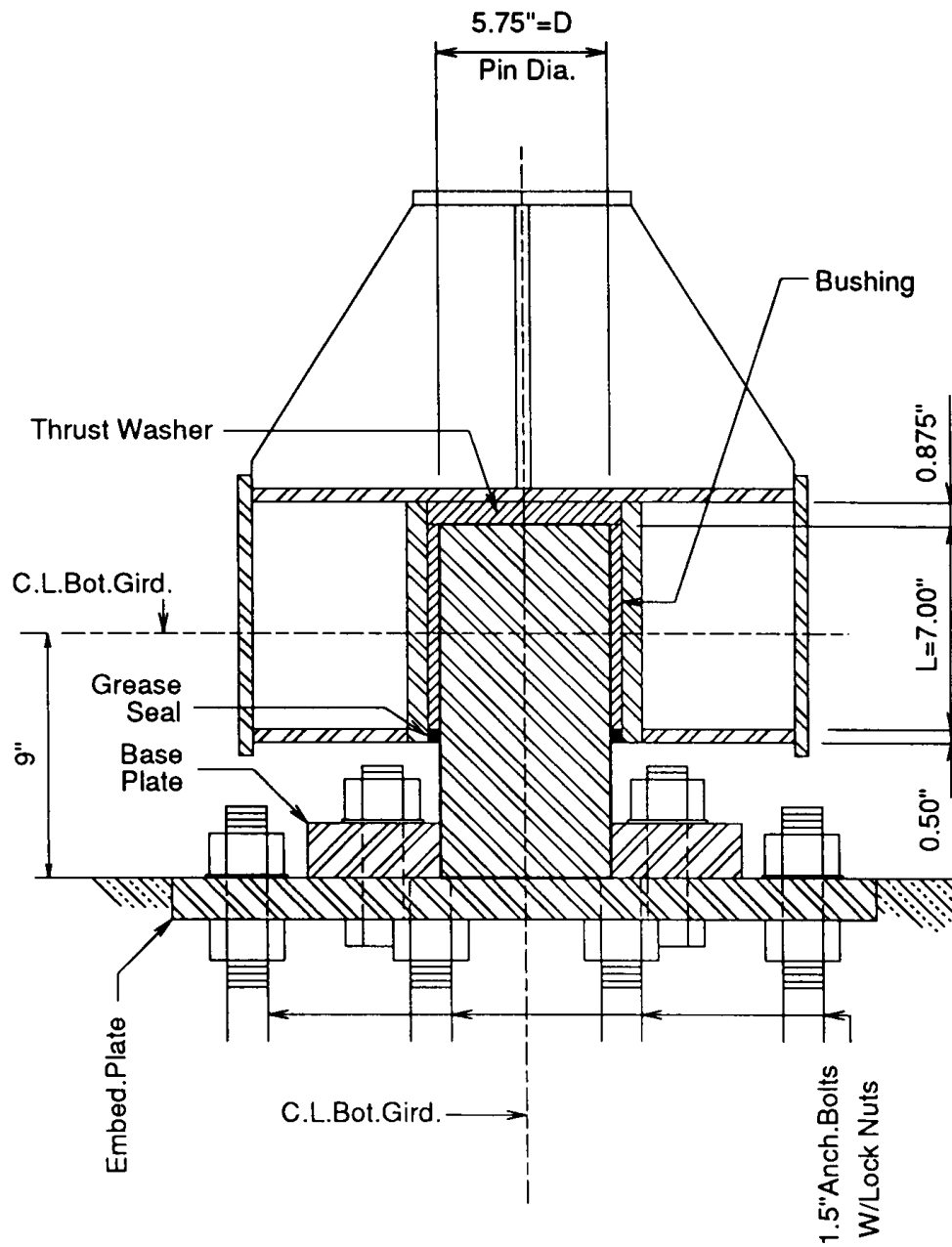
5.75"=D

Pin Dia.

Bushing

Thrust Washer

0.875"

C.L.Bot.Gird.

L=7.00"

Grease
Seal

9"

Base
Plate

0.50"

Embed.Plate

C.L.Bot.Gird.

1.5"Anch.Bolts
W/Lock Nuts

SECTION THRU BOTTOM HINGE

CALCULATION FOR ACTUAL MAXIMUM BEARING PRESSURE-BOTTOM PIN

L = Bushing Length = 7", Nominal Pin Diameter = 5.75"

R1 = Min. Radius Of Pin = 2.8670"

R2 = Max. Inside Radius Of Bushing = 2.8715"

E1 = Modulus Of Elasticity Of Pin = 29000ksi

E2 = Modulus Of Elasticity Of Bushing = 15000ksi

P1 = 115.49/7 = 16.50k/in

$$f_{p\ max.} = 0.591 \sqrt{\frac{P1\ E1\ E2\ (R2\text{-}R1)}{(E1\text{+}E2)(R1R2)}}$$

$$f_{p\ max.} = 0.591 \sqrt{\frac{16.5 \times 435 \times 10^6 \times 0.0045}{44 \times 10^3 \times 8.25843}} = 5.572ksi$$

A Maximum Bearing Stress Equal To Or Less Than The Yield
Strength Of The Material Is Allowable.

TOP HINGE:

PLAN

SECTION A-A
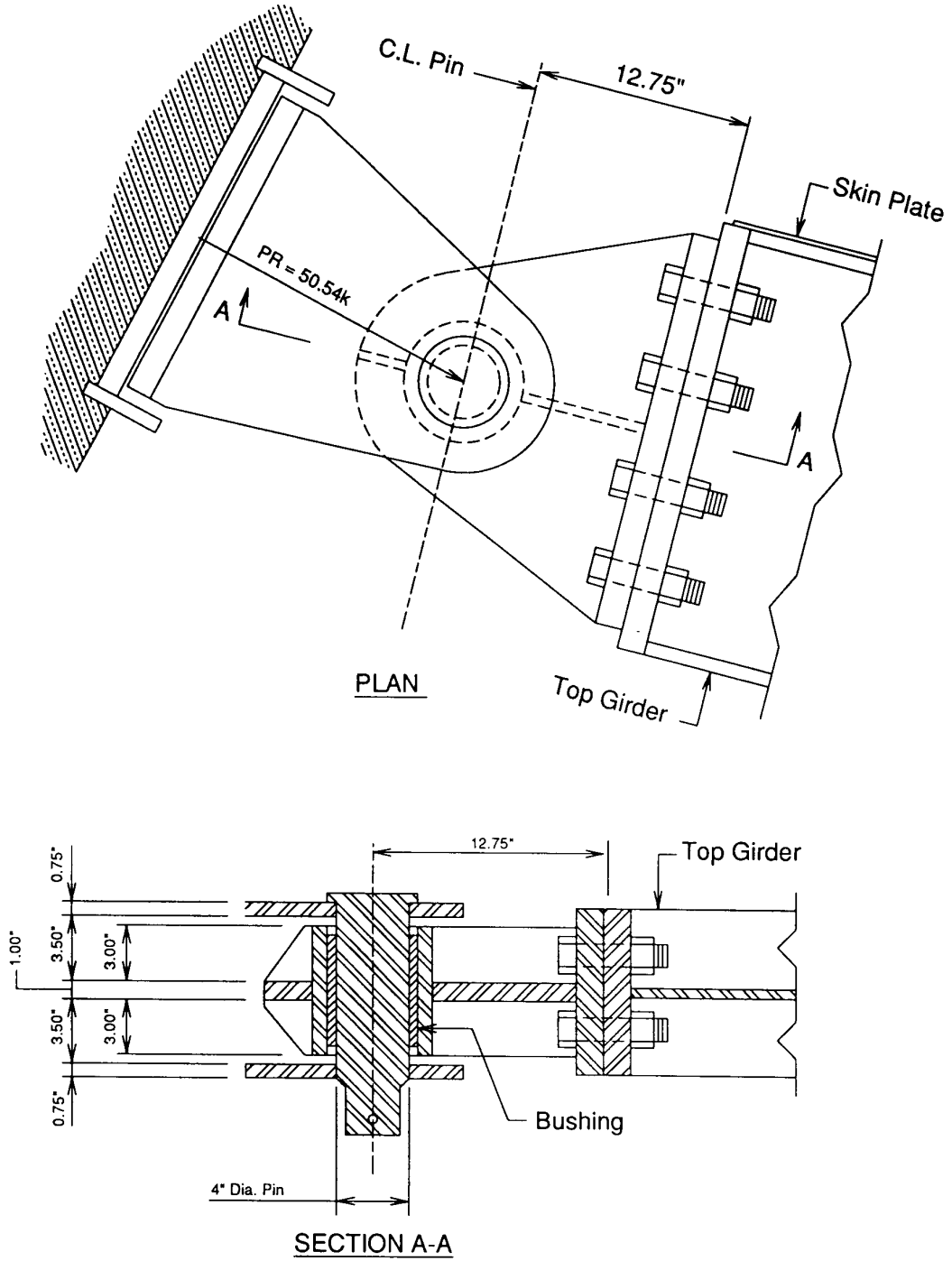
<u>TOP PIN</u> (Mat'l. ASTM A276, Fy = 55ksi)

P_R = 50.54k, V = 50.54/2 = 25.27k (Dbl. Shear)

M = 50.54x8.75/4 = 110.56k-in

A = 3.1416(4)^2/4 = 12.57, S = 3.1416(4)^3/32 = 6.28

F_b=1.11(0.75F_y)=45.83ksi

F_V = 1.11(0.4F_y) = 24.44ksi

f_V = 25.27/12.57 = 2.01ksi < 24.44ksi

f_b= 110.56/6.26 = 17.61ksi < 45.83ksi

<u>TOP BUSHING</u> (Mat'l. ASTM B22)

P_R = 50.54k, Bushing Length = 6.00"

Inside Dia. = 4.00"

F_p = Avg. Allowable Brg. Stress =1.11x3.00 = 3.33ksi

f_p = 50.54/(4x6) = 2.11ksi < 3.33ksi

CHECK MAX. BEARING STRESS:

R1 = 1.997", R2 = 2.001"

E1 = 29000ksi, E2 = 15000ksi

P1 = 50.54/6 = 8.42k/in

$$(Max) \; f_p = 0.591 \sqrt{\frac{8.42 \times 29 \times 15 \times 10^6 (2.001 - 1.997)}{(29 + 15)(10^3)(2.001 \times 1.997)}} = 5.396ksi$$

A Maximum Bearing Stress Equal To Or Less Than The Yield Strength Of The Material Is Allowable.

Appendix E
Rolling Gate Design Example

The opening for the closure in this example is 60 ft wide by 9.63 ft high.

Load Cases:

In accordance with EM 1110-2-2502, consideration shall be given to load cases I1 through I4.

Case I1, Design Flood Loading. Gate is closed; water on the unprotected side is at the design flood elevation; water is at or below the sill surface on the protected side. When skin plate is on the unprotected side, stability against flotation shall be checked. Design stresses shall not exceed 5/6 of the stresses allowed in AISC (1989).

Case I2, Maximum Flood Loading. Same as case I1 except that water level is to top of gate on unprotected side. Design stresses shall not exceed 1.11 times the stresses allowed in AISC (1989).

Case I3, Earthquake Loading. This case is applicable to support structures only.

Case I4, Short-Duration Loading. Gate is in any position and is subjected to construction and/or wind loads. Design stresses shall not exceed 1.11 times the stresses allowed in AISC (1989).

In this example, cases I1 and I3 are not significant, and skin plate, intercostals, and girders are designed for case I2. Case I4 is applicable for design of latching devices and for checking overturning stability for wind loading.

The skin plate is designed as a fixed end member spanning between intercostals. In order to ensure that the flat plate theory is applicable, deflection will be limited to 0.4 of plate thickness.
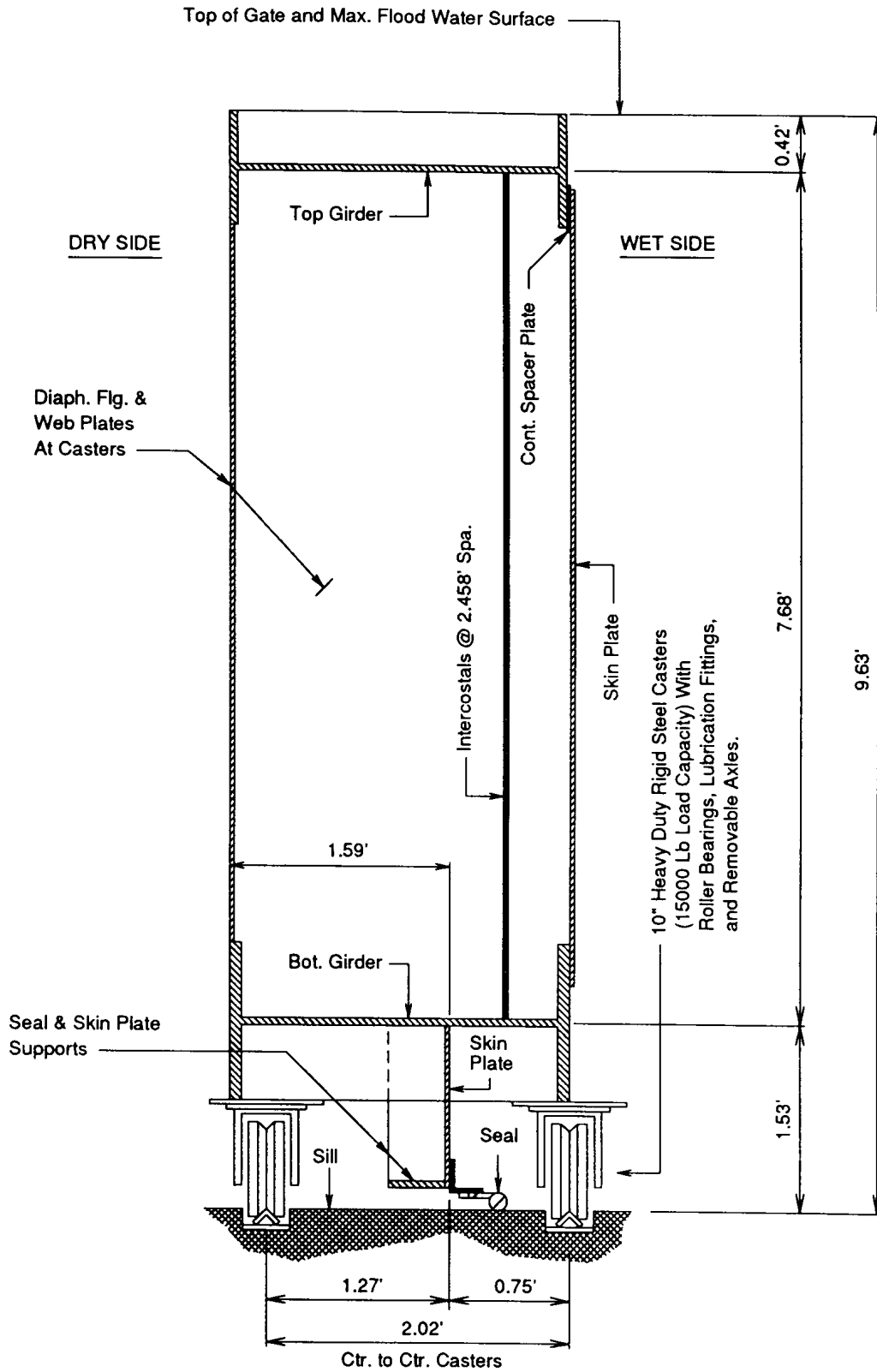
The intercostals are designed as simple beams spanning between girders.

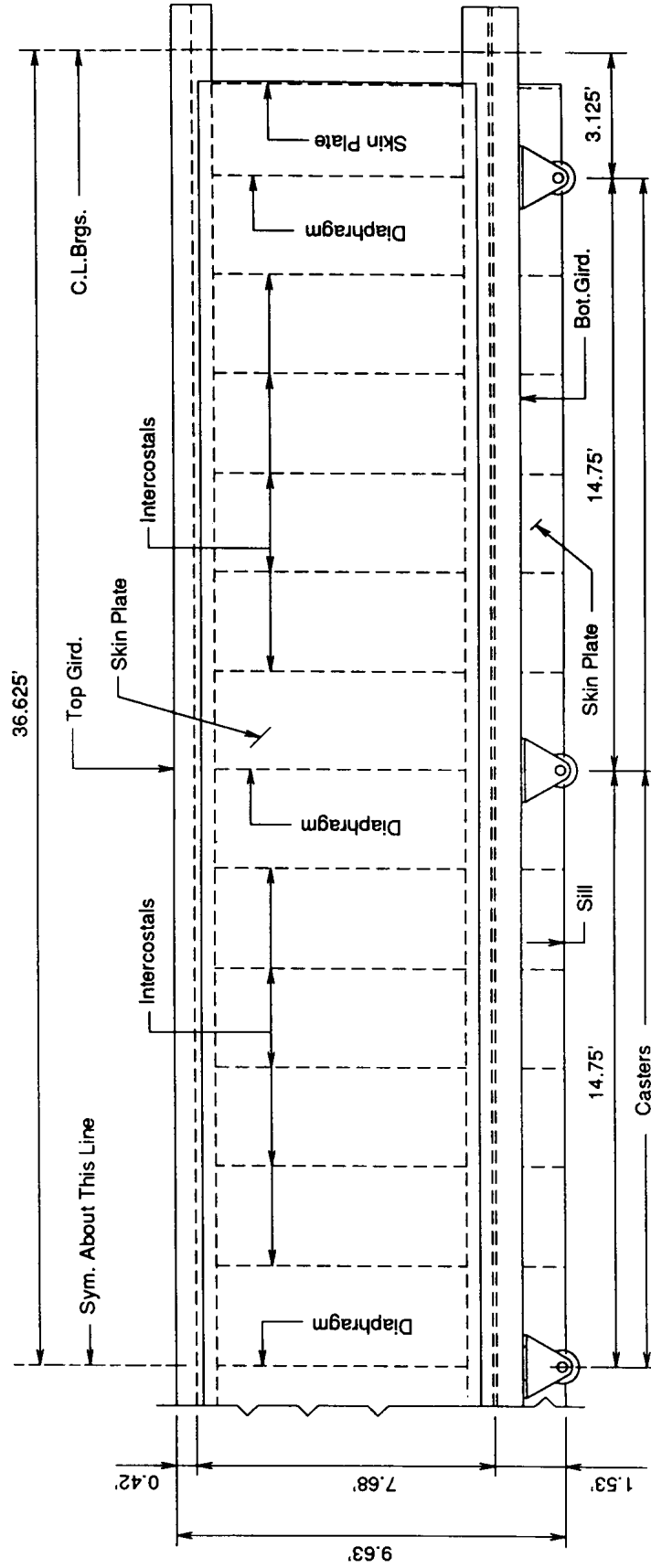Girders are designed as simple beams spanning between bearing blocks on each side of the opening.
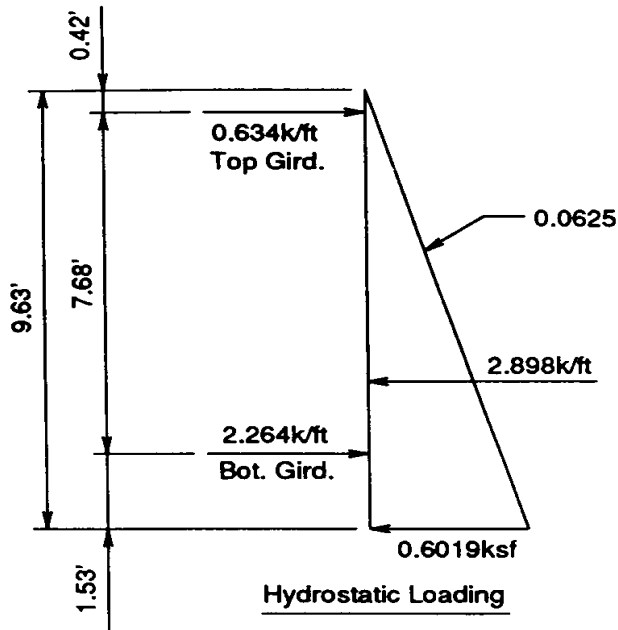
HORIZONTAL SECTION THRU GATE

Top of Gate and Max. Flood Water Surface

DRY SIDE

WET SIDE

Top Girder

Cont. Spacer Plate

0.42'

Diaph. Flg. &
Web Plates
At Casters

Intercostals @ 2.458' Spa.

Skin Plate

10" Heavy Duty Rigid Steel Casters
(15000 Lb Load Capacity) With
Roller Bearings, Lubrication Fittings,
and Removable Axles.

7.68'

9.63'

1.59'

Bot. Girder

Seal & Skin Plate
Supports

Skin
Plate

Sill

Seal

1.53'

1.27'

0.75'

2.02'

Ctr. to Ctr. Casters
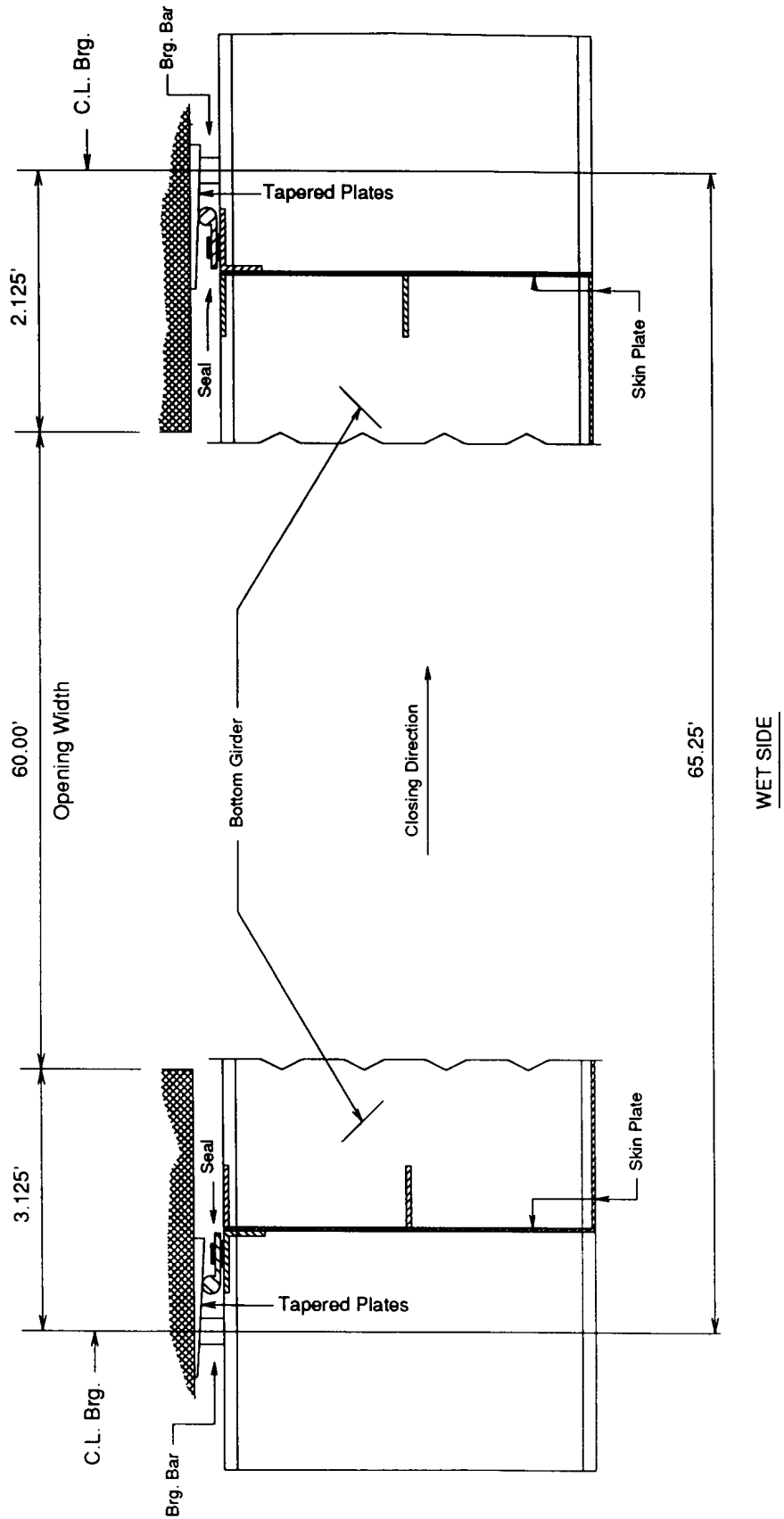
VERTICAL SECTION THRU GATE

1/2 WET SIDE ELEVATION

GIRDER DESIGN (Mat'l. ASTM A36)

Hydrostatic Loading

0.42'

9.63'

7.68'

1.53'

0.634k/ft
Top Gird.

0.0625

2.898k/ft

2.264k/ft
Bot. Gird.

0.6019ksf

Bottom Girder:

Trial Section W27x194

$A=57$, $I=7820$, $S=556$

$W=2.264k/ft$

$L=65.25'$

$M=WL^2/8=1204.89k\text{-}ft$

$M=14458.68k\text{-}in$

Since compression flange is braced continuously and compact section requirements of Table B5.1 of AISC (1989). are satisfied, the allowable bending stress is:

$F_b =1.11(2/3)F_y = 26.67ksi$

$f_b =M/S=14458.68/556=26.00ksi<F_b =26.67ksi$

USE: W27x194 FOR BOTTOM GIRDER

Top Girder:

Trial Section W27x84, $A=24.8$, $I=2850$, $S=213$

$W=0.634k/ft$, $L=65.25'$, $M=0.634(65.25)^2/8=337.41k\text{-}ft$

$M=4048.94k\text{-}in$

$f_b =M/S=4048.94/213=19.01ksi<F_b =26.67ksi$

USE: W27x84 FOR TOP GIRDER

SKIN PLATE DESIGN   (Mat'l. ASTM A36)

p (6" above girder flg.) $=0.0625(9.63-1.53-0.58-0.5)=0.439$ksf

$p = 0.00305$ksi, $L=2.458'=29.5"$, $M=pL^2/12=0.221$k-in

$F_b = 1.11(0.75)F_y = 30$ksi

t_{min} for stress $=(6M/F_b)^{0.5}=(6\times0.221/30.0)^{0.5}=\underline{0.2102"}$

t_{min} for defl $=(pL^4/12.8E)^{0.25}$

t_{min} for defl $=[0.00305(29.5)^4/(12.8\times29000)]^{0.25}=\underline{0.2809"}$

USE 5/16" SKIN PLATE

INTERCOSTAL DESIGN   (Mat'l. ASTM A36)

1.760k   0.0645k/ft

y

7.68'

3.266k   1.2444k/ft
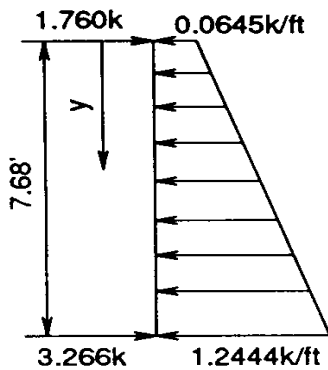
Hydrostatic
Loading

$M_y = 1.76y - 0.03225y^2 - 0.025605y^3$

$V_y = 1.760 - 0.0645y - 0.0768y^2$

$V_y = 0$ and M_y is maximum at $y=4.386'$

$M_{max} = 4.939$k-ft$=59.27$k-in

Trial Section WT7x13, S=3.31

$f_b = M/S = 59.27/3.31 = 17.91$ksi

$F_b = 1.11(0.6)F_y = 24$ksi > 17.91ksi

The WT7x13 is able to support the load without considering the contribution of the skin plate.

USE WT7x13 FOR INTERCOSTALS

CHECK RESISTANCE TO FLOTATION
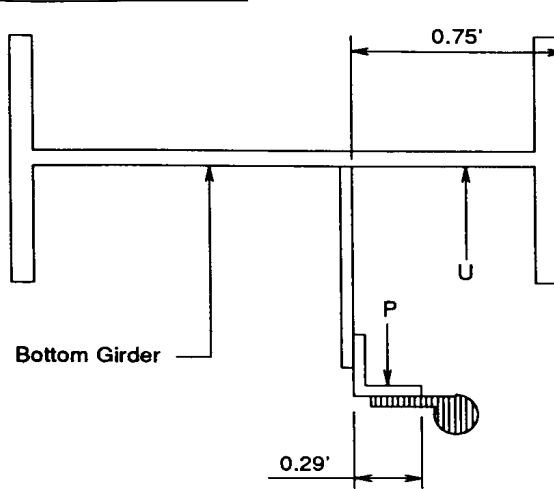
U=0.75(0.0625)(8.16)=0.3825k/ft , W=Wt. of Gate=0.4200k/ft

P=0.29(0.0625)(9.44)=0.1711k/ft , FS=Factor of Safety

FS=(W+P)/U=(0.4200+0.1711)/0.3825=1.55

Factor of Safety Req'd. by Ref. 3.a.(6). = 1.5

FS=1.55>1.5   o.k.

CHECK WIND LOAD STABILITY

Wheel Loads
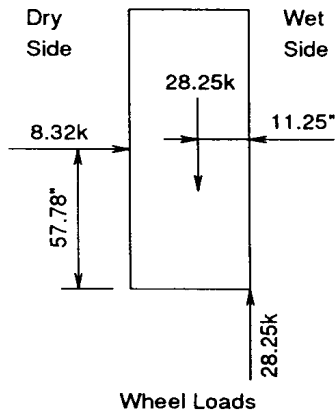
Assume gate is closed, but unlatched, and subjected to a 75-MPH wind load.
Wind Pressure=$0.00256(75)^2$ =14.4psf
Wind Load=0.0144(60)(9.63)=8.32k
Gate Weight=28.25k

FS(overturn)=28.25x11.25/(8.32x57.78)

FS=0.66<1.00

Gate Must Be Latched To Withstand A 75-MPH Wind.

GATE MUST ALWAYS BE LATCHED WHEN IN CLOSED POSITION

E-7

Appendix F
Trolley Gate Design Example

The opening for the closure in this example is 32 ft wide by 24.5 ft high.

Load Cases:

Load cases used are for inland floodwalls in accordance with EM 1110-2-2502. Load case I1 was found to be critical for design of the lower girder and the skin plate. Load case I2 was found to be critical for design of the top girder and the intercostals. The maximum wind speed that may be allowed when the gate is not in a latched condition was calculated; this is a short-duration load similar to Case I4. The Trolley Rail Support Beam was designed for the stresses allowed in AISC (1989), with the total deflection limited to 1/700 of the span.

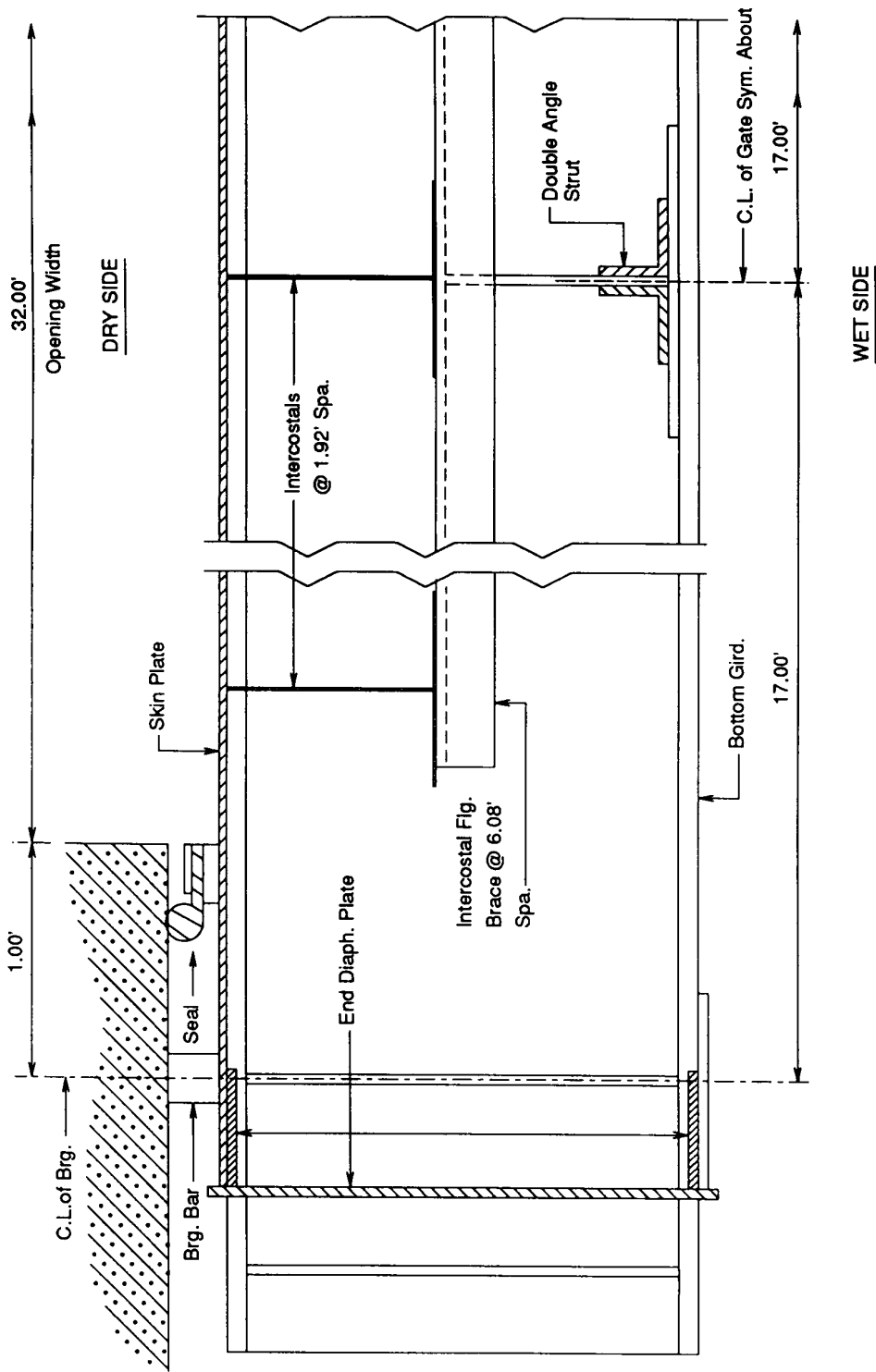Case I1, Design Flood Loading. Gate is closed; water on the unprotected side is at the design flood elevation (in this example 3 ft below top of gate); water is at or below sill surface on the protected side. Design stresses shall not exceed 5/6 of the stresses allowed in AISC (1989).

Case I2, Maximum Flood Loading. Same as case I1 except that water level is to top of gate on unprotected side. Design stresses shall not be greater than 1.11 times the stresses allowed in AISC (1989).
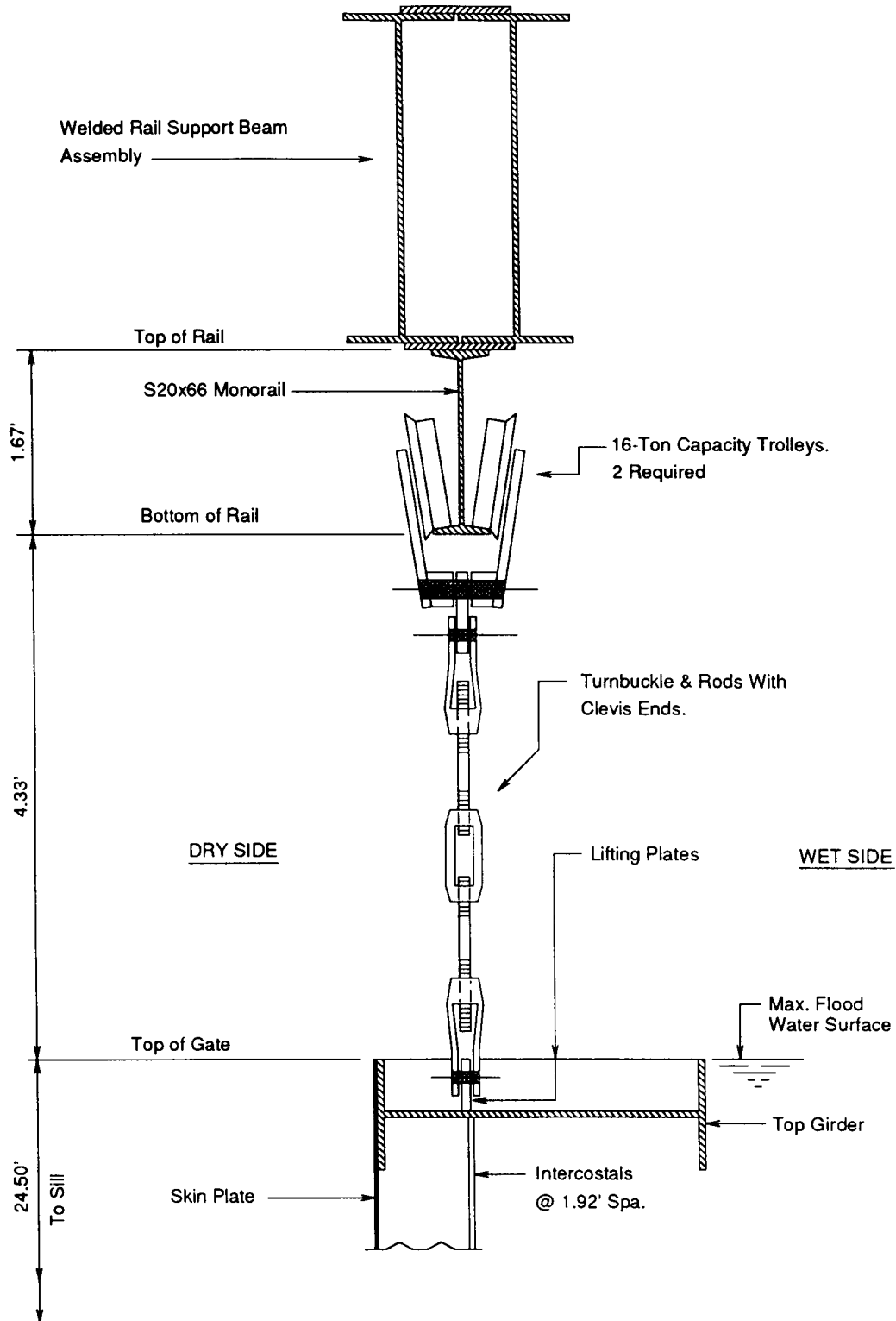
The skin plate is designed as a fixed end beam, spanning between intercostals. In order to ensure that the flat plate theory is applicable, deflection is limited to 0.4 of thickness.

The intercostals are designed as simple beams spanning between girders.

Girders are designed as simple beams spanning between bearings on each side of the opening.
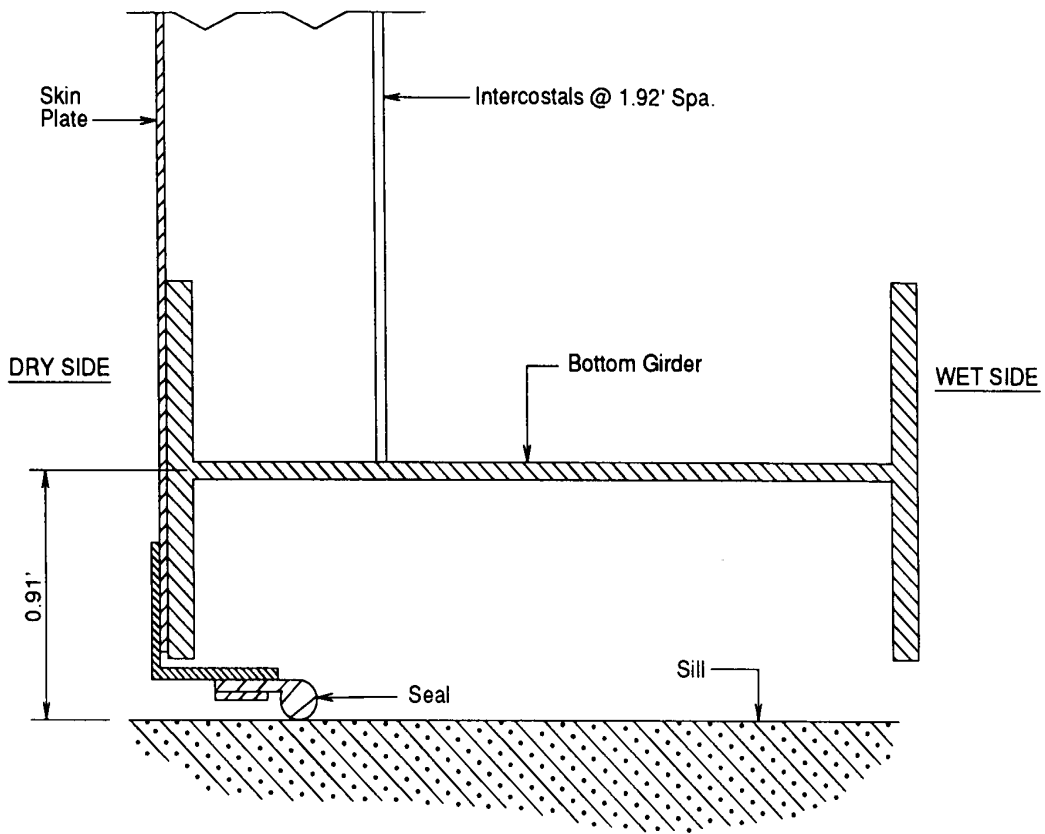
1/2 HORIZONTAL SECTION THRU GATE

32.00'

Opening Width

DRY SIDE

WET SIDE

C.L. of Gate Sym. About

17.00'

17.00'

Double Angle Strut

Intercostals @ 1.92' Spa.

Skin Plate

Bottom Gird.

Intercostal Flg. Brace @ 6.08' Spa.

End Diaph. Plate

1.00'

Seal

C.L. of Brg.

Brg. Bar

Welded Rail Support Beam
Assembly

Top of Rail

S20x66 Monorail

16-Ton Capacity Trolleys.
2 Required

1.67'

Bottom of Rail

Turnbuckle & Rods With
Clevis Ends.

4.33'

DRY SIDE

Lifting Plates

WET SIDE

Max. Flood
Water Surface

Top of Gate

Top Girder

24.50'
To Sill

Intercostals
@ 1.92' Spa.

Skin Plate

VERTICAL SECTION THRU TOP OF GATE
AND TROLLEY SUPPORT

Skin
Plate

Intercostals @ 1.92' Spa.

DRY SIDE

Bottom Girder

WET SIDE

0.91'

Seal

Sill

VERTICAL SECTION THRU BOTTOM GIRDER
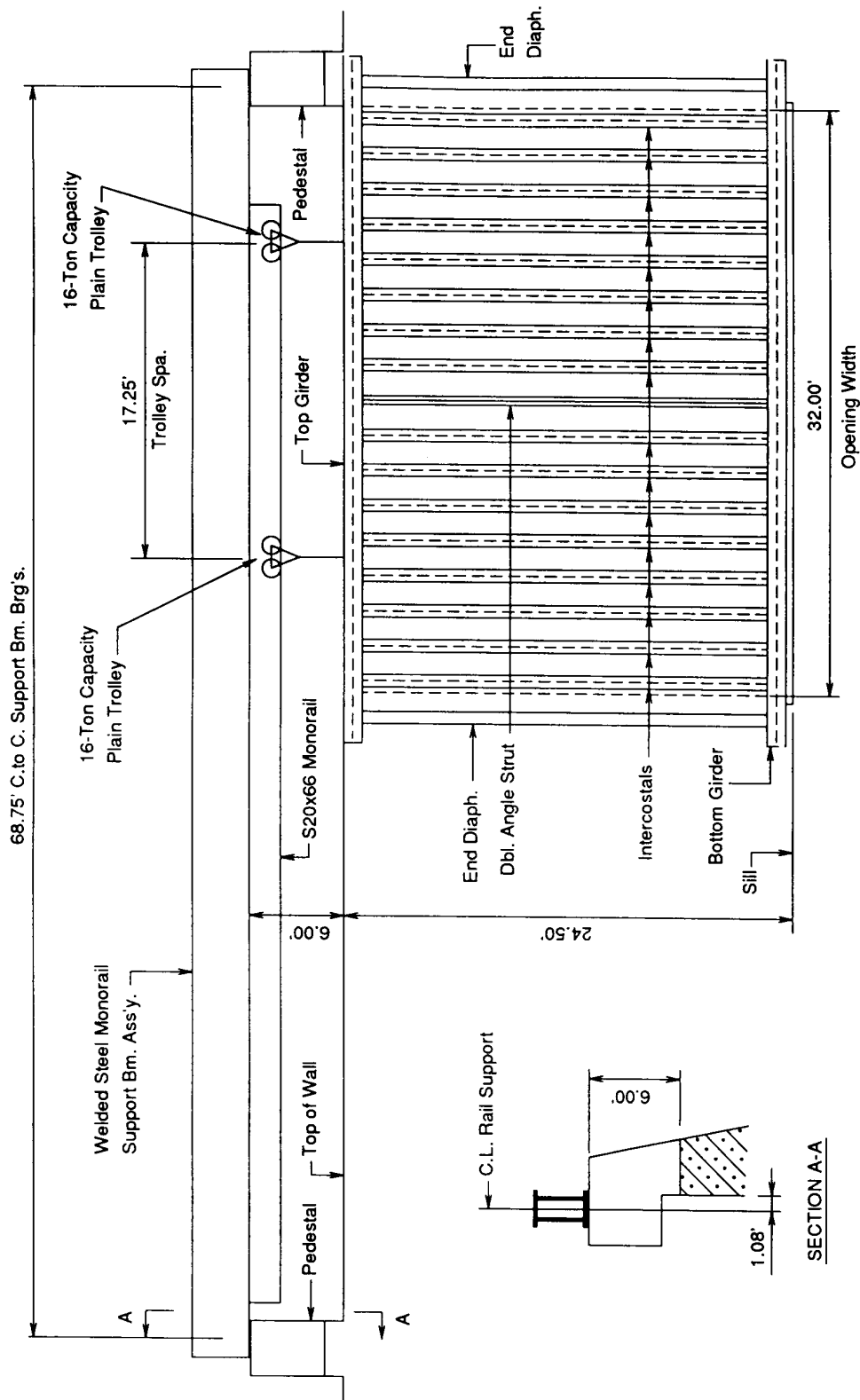
WET SIDE ELEVATION

GIRDER DESIGN
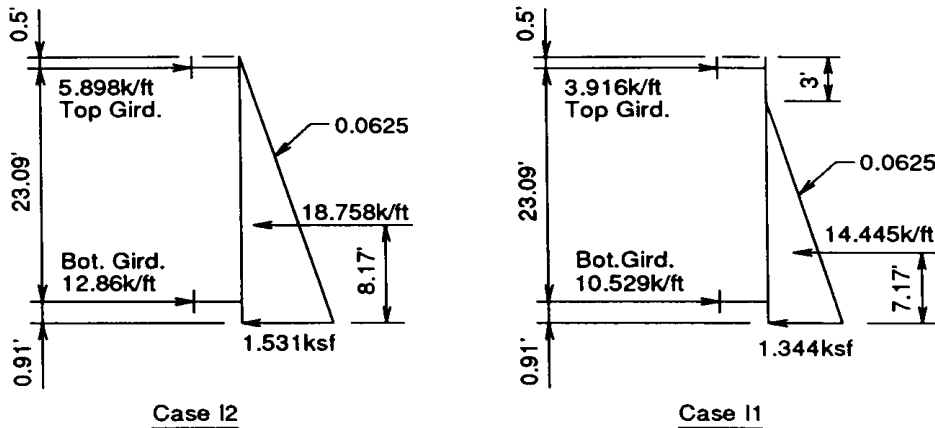
Case I2   Case I1

Girder Loadings

By inspection of the above loadings, design of the bottom girder
is governed by Case I1 and design of the top girder is controlled
by Case I2.

BOTTOM GIRDER: Eq. & Table No's. Are From AISC (1989).

Material ASTM A572-Gr.50, Trial Section W36x230: S=837, r_T =4.3

b_f =16.50", L_b =17'=204", F_y =50ksi, L_c =76(16.50)/(50)$^{1/2}$ (F1-2)

L_c =177.34"<L_b =204", L_b /r_T =47.44

F_b =[2/3-50(47.44)^2 /(1530x10^3)]50x5/6=24.72ksi   (F1-6)

M=10.529(34)^2 /8=1521.44k-ft=18257.28k-in

f_b =M/S=18257.28/837=21.81ksi<24.72ksi

> USE W36x230 FOR BOTTOM GIRDER, A572-GR.50 STEEL

TOP GIRDER: Eq. & Table No's. Are From AISC (1989).

Material ASTM A572-Gr.50, Trial Section W36x135:S=439, r_T =2.93

b_f =12", L_b =204", L_c =76b_f /(F_y)$^{1/2}$ =129<204   (F1-2)

L_b /r_T =69.62

F_b =[2/3-50(69.62)^2 /(1530x10^3)]50x1.11=28.21ksi   (F1-6)

M=5.898(34)^2 /8=852.26k-ft=10227.1k-in

f_b =M/S=10227.1/439=23.30ksi<28.21ksi

> USE W36x135 FOR TOP GIRDER, ASTM A572-GR.50 STEEL

SKIN PLATE

p=water pressure 6" above bottom girder flange.

Skin plate stresses are critical for Case I1

$p=0.0625(21.5-0.91-8.25/12-0.5)=1.213ksf=0.00842ksi$

$L=1.92'=23", M=pL^2/12=0.00842(23)^2/12=0.3712k\text{-in}$

$F_b=(5/6)(3/4)36=22.50ksi, t_{min\text{-}stress}=(6M/F_b)^{1/2}$

$t_{min\text{-}stress}=(6\times0.3712/22.5)^{1/2}=\underline{0.3146}"$
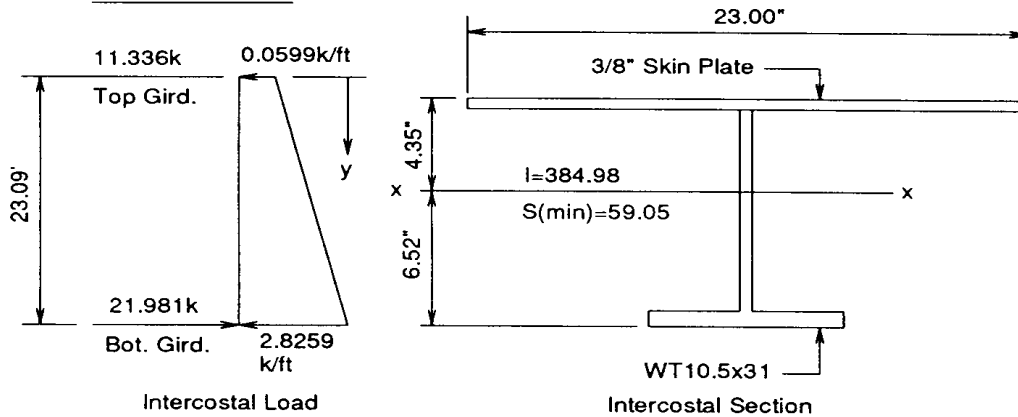

Maximum deflection will occur under Case I2.

$p=0.0625(24.5-0.91-8.25/12-0.5)=1.400ksf=0.00972ksi$

For deflection equal to 0.4t, $t_{min\text{-}defl}=(pL^4/12.8E)^{1/4}$

$t_{min\text{-}defl}=[0.00972(23)^4/(12.8\times29000)]^{1/4}=\underline{0.2926}"$

USE 3/8" SKIN PLATE, ASTM A36

INTERCOSTALS

Intercostal Load

Intercostal Section

Intercostal Stresses Are Critical For Case I2.

$M_y=11.336y-0.0300y^2-0.02y^3$, M_y is maximum at $y=13.26'$
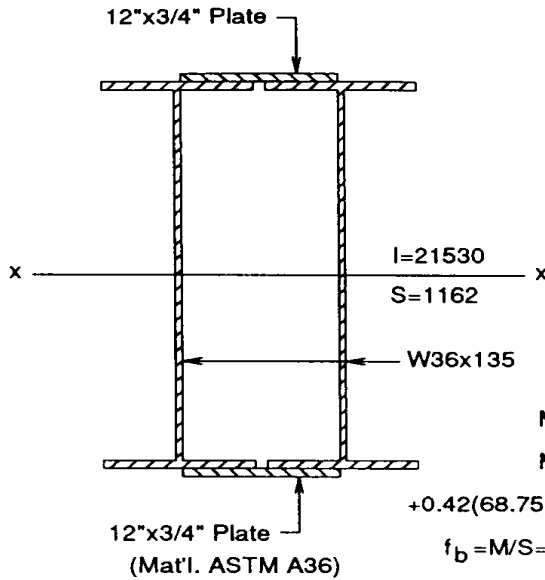
$M_{max}=98.41k\text{-ft}=1180.92k\text{-in}$

$L_b=6.08'=73", L_c=76b_f/(F_y)^{1/2}=104.37">73"$ (F1-2)

Section is compact. $F_b=1.11(2/3)F_y=1.11(2/3)(36)=26.67ksi$

$f_b=M/S=1180.92/59.05=20.00ksi<26.67ksi$

USE WT10.5x31 INTERCOSTALS, ASTM A36

MONORAIL SUPPORT BEAM

12"x3/4" Plate

$I=21530$

$S=1162$

W36x135

12"x3/4" Plate
(Mat'l. ASTM A36)

Rail Support Section

Gate Wt.=49k

P=Trolley Load=49/2=24.5k

I=Impact=P/4=6.125k

P+I=30.625k

W=Wt.Supp't.Bm.& Rail=0.42k/ft

L=Span =68.75'

a=Trolley Spacing=17.25'

$M=(P+I)(L-a/2)^2/(2xL)+WL^2/8$

$M=30.625(68.75-8.625)^2/137.5$

$+0.42(68.75)^2/8=1053.33\text{k-ft}=12640\text{k-in}$

$f_b=M/S=12640/1162=10.88\text{ksi}$ o.k.

understressed

CHECK DEFLECTION:

L=68.75'=825", a=17.25'=207", W=0.42k/ft=0.035k/in

Live Load Defl.= $\triangle_{LL}=P(L^3-3a^2L+2a^3)/24EI$

$\triangle_{LL}=24.5[(825)^3-3(207)^2(825)+2(207)^3]/(24\times29000\times21530)=$

$=0.774"$, $\triangle_{LL}/L=0.774/825=1/1066<1/1000$ o.k.

Total Defl.=$0.774"+5WL^4/384EI=\triangle_T$

$\triangle_T=5(0.035)(825)^4/(384\times29000\times21530)+0.774"=1.112"$

$\triangle_T/L=1.112/825=1/742<1/700$ o.k.

Size of Section is Controlled By Deflection.

USE 2-W36x135 With 12"x3/4" Plate Top & Bottom

CAMBER BEAM FOR TOTAL DEFLECTION

GATE HANGERS

$P+I=30.625k$

> Use 1.50" Dia. Rods with 2" Dia. Upset Threaded Ends, Number
> 5 Clevises with 2" Dia. Pins, and Standard Turnbuckles.

Material: ASTM A36.

Safe Load For Clevises=37.5k>30.625k

Safe Load For Turnbuckles=37.2k>30.625k

ROD TENSION:

A_R =Cross Sectional Area of Rod=1.767

A_T =Cross Sectional Area at Threads=2.50

F_t =Allowable Tension=18ksi

f_t (rod)=30.625/1.767=17.33ksi<18ksi

f_t (thread)=30.625/2.50=12.25ksi

Note: The maximum angle (from vertical) that the gate may
rotate without inducing moment in the hangers is 2.8624 degrees.
This is determined from the tolerances between the clevis and the
lifting plate at the top of the gate.

W=Weight of Gate=49k, P=Horizontal Wind Load.

Y_p =Distance below clevis to line of action of wind load. =12.0'

Y_w=Distance below clevis of gate C.G.=12.73'

A_o =Gate opening area=32x24.5=784Sq.Ft.

X=Horizontal displacement of gate C.G.=Y_w Sin(2.8624)=0.6357'

p=Wind load=0.00256V^2 , V=Wind speed in MPH.

P=Wind load=pA_o =WX/Y_p =49000(0.6357)/12.00=2595.78

p=2595.78/A_o =3.311psf

V^2 =p/0.00256=3.311/0.00256=1293.36, V=35.96 MPH.

> This indicates that gate must be latched in position for wind
> speeds in excess of 35MPH.

GATE MUST ALWAYS BE LATCHED WHEN IN CLOSED POSITION